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The Injury to the Hull of the U. S. S. "Arkansas" by Grounding

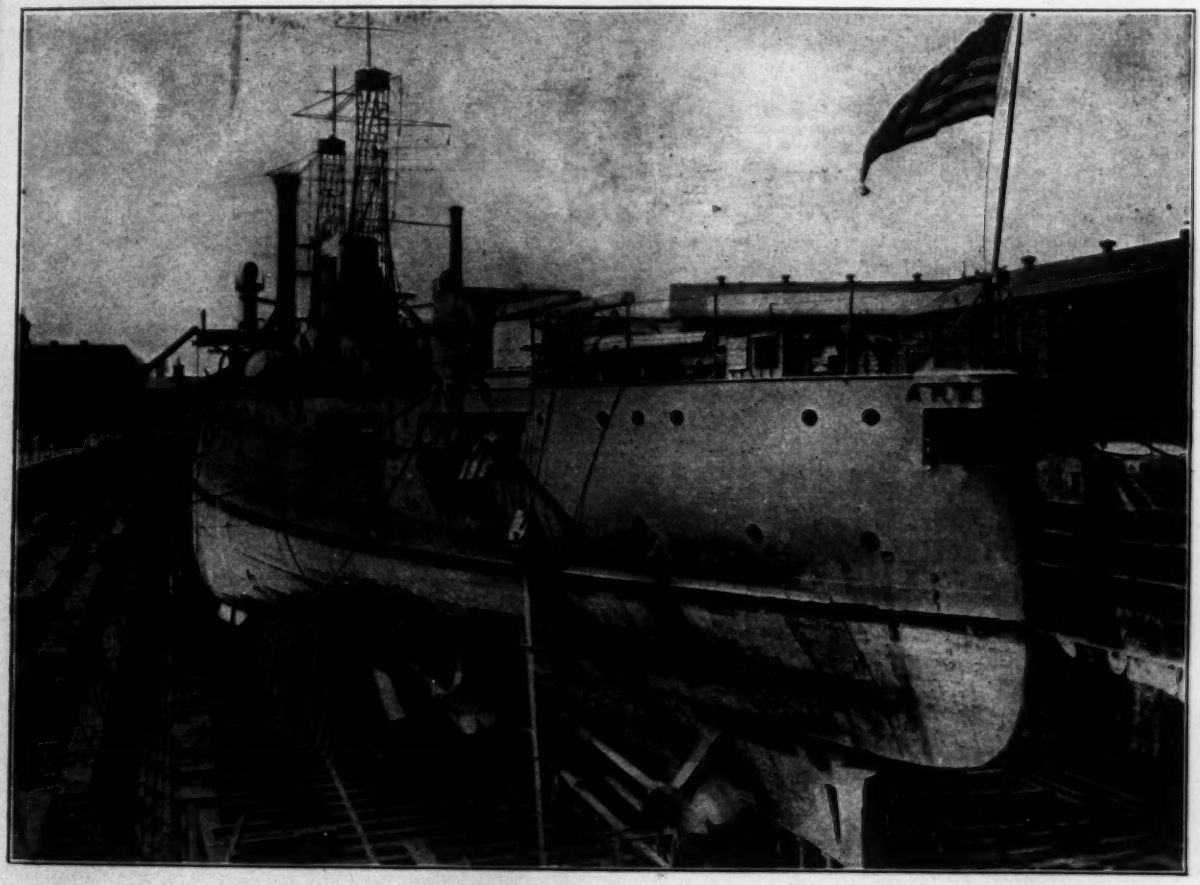
(CONTRIBUTED)

In the course of the discussion consequent on the foundering of the "Titanic," frequent occasion was taken to compare the methods of construction and water-tight division of warships to those in that unfortunate vessel. It was stated freely that, had an accident, similar to that which overcame the "Titanic" occurred to a man-of-war, the consequences would have been comparatively insignificant and the ship would have been able to make port without even disembarking passengers.

In view of all this, it seems a remarkable coincidence that within less than a year one of the newest battleships, representing the last word in warship construction, as did the "Titanic" in merchant construction, should experience an accident of the same general kind. While it is not possible to draw accurate conclusions as to what might have happened to either ship under the circumstances attending the accident to the other, yet a description of the damages to the U. S. S. "Arkansas" and some comparisons at least should prove of interest.

As well will be remembered, the generally accepted theory of the accident to the "Titanic" is that a projecting underwater ledge of the iceberg ripped open the shell plating of the ship for about 200 ft. and permitted water to enter the six forward compartments. As these compartments filled, the vessel sank by the head and water flowing over the tops of the water-tight bulkheads reached, successively, compartments farther aft until finally the vessel, going down deeper by the head, sank bow on.

The accident to the "Arkansas" occurred at six o'clock



U. S. S. "ARKANSAS" IN DRY DOCK AT THE BROOKLYN NAVY YARD FOR REPAIRS TO THE BOTTOM
(Showing special shoring to support overhang during work on the under frame.)

in the evening of Feb. 11, 1913, while the vessel was proceeding at a speed of 12 knots from Guaycanayabo Bay, Cuba, to Guantanamo. Although the chart indicated over 60 ft. of water, and soundings taken just before and after the accident showed this depth, except in one spot, yet the vessel struck bottom when drawing slightly over

FIG. 2. STARBOARD SIDE LOOKING AFT

FIG. 3. PORT SIDE LOOKING AFT

FIGS. 2 AND 3 DAMAGE TO THE BOTTOM OF THE "ARKANSAS"

25 ft. forward, 28 ft. aft. The obstructions undoubtedly consisted of uncharted coral pinnacles. These apparently were broken off by the ship riding over them, as the damages decreased toward the stern even though the ship drew more water aft than forward. The "Arkansas," in striking, rolled first to one side and then to the other and there was a succession of shocks, compared by one officer to those experienced when a street car runs off the track and over cobble stones.

The discipline on board was excellent; the crew responded promptly to the call for "collision quarters" and all water-tight doors and hatches were closed until examination could be made to determine the extent of the damages. It was found that some twelve of the double-bottom compartments were leaking, though the water in several of these could be kept down by the pumps. The inner bottom held and the leakage was confined practically to those compartments that were opened to the sea by the damage to shell plating, and which were of comparatively small volume. The ship was able to continue on her trip and later came north, without convoy, for repairs; at no time was there the slightest danger of foundering as a result of the accident.

The vessel came to the Brooklyn Navy Yard and on Mar. 1, nearly four weeks after the accident, was floated into dry dock No. 4. When the bottom of a heavy ship has been damaged in any way, the usual difficulties attendant on dry docking are greatly increased. The weight of the ship, which is supported, when afloat, by the buoyancy of the water, must be concentrated when in dry dock and supported on the bottom plating and framing. When damage to a ship's bottom has been sustained, any of the plating or framing that has buckled cannot be called on to support its share of the weight. Fortunately enough, in the case of the "Arkansas," as the event proved, the framing directly in way of main keel and docking keels, which rest on the blocks in the dry dock, was not damaged. The only serious danger lay, therefore, in the possibility of further distortion of the structure due to the weight of the overhanging portions of the ship, which depend normally when in dry dock upon girder strength of the structure for their support. This weight was taken care of on the "Arkansas," as shown in Fig. 1, by shoring fitted as soon as possible after the water had been pumped out of the dock.

After the "Arkansas" had been floated into the dry dock and enough water pumped out to land her on the blocks, examination was made by a diver to find if the vessel was resting properly. It was found that for a considerable distance forward she was not bearing on the blocks; though at first it appeared that this was due to local deformation of the docking keel framing, later it developed that such was not the case and that it was due in all probability to a slight deformation of the ship as a whole. As the water was pumped out of the dock, further examination revealed that more and more of the surface was bearing as the weight came on the blocks, until finally when the dock was dry the main and docking keels bore on the blocks properly throughout their whole length, though the weight was not equally distributed.

Examination of the damage, made after the dock was

FIG. 4. ONE OF THE HOLES IN THE BOTTOM OF THE "ARKANSAS"; STARBOARD SIDE LOOKING AFT

dry, showed that one pinnacle rock struck the bottom just under the port forward bilge keel and distorting this badly, forced in the shell plating in way of it. Another rock scraped along the bottom on the port side for over 200 ft., and for that distance made an indentation simi-

lar to an inverted trough, opening seams, tearing holes in the shell plating and buckling framing, brackets and longitudinals. On the starboard side, a rock struck and caused similar damages for a distance of over 100 ft. In other places the ship apparently rode over rocks which, however, caused no damage other than slight buckling of framing. That the damage sustained by the vessel as a whole was so comparatively small, may be attributed to the excellent material and workmanship in the ship and the rigidity of the structure as designed.

The repairs required were considerable in extent and involved removal of 28 shell plates, of which 12 were so badly damaged as to require to be replaced with new ones, while the remainder permitted of being straightened and replaced. Internal framing and brackets were badly buckled and broken throughout the area of the damage and had to be repaired by cutting out and renewing the damaged portions. While this work was in progress pre-

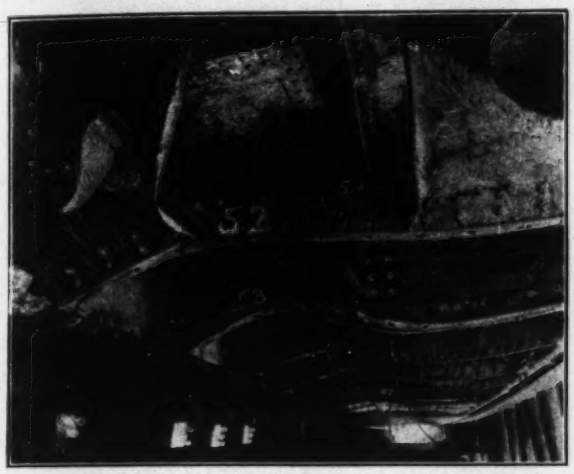


FIG. 5. BOTTOM OF THE "ARKANSAS" (PORT SIDE LOOKING AFT) AFTER REMOVAL OF SHELL PLATING, SHOWING BENT AND BROKEN FRAMES AND FLOOR PLATES

cautions had to be taken to guard the integrity of the structure as a whole against deformation when portions were removed for repairs. In a number of locations where there was slight buckling of brackets plates these were straightened and strengthened by riveting on angle stiffeners.

While it does not seem safe to conclude that the "Titanic" would have sunk had she experienced a similar accident to that of the "Arkansas," yet it does seem justifiable to say that the "Arkansas" would have been safe had she had the same experience as the "Titanic." Though the latter vessel was proceeding at about twice the speed of the "Arkansas," the force of the blow experienced by each can be compared by the sensations of those on board. No one on the "Titanic" seems to have felt anything unusual, while on the "Arkansas" everyone felt the shocks as the ship rolled and bumped when her framing struck the rocks and rode over them.

The "Titanic" had an inner bottom which, however, did not extend above the turn of the bilges, while the "Arkansas" was so fitted up to the waterline. There is no reason to suppose that it would have proved less efficacious in case of a side blow such as the "Titanic" experienced, than it did for the damage actually sustained by the "Arkansas," which was on the flat bottom. The efficacy and value of an inner bottom over all the submerged portions of a ship are demonstrated and ample justification is given, if any had been needed, for the recent action of the owners of the "Olympic" in extending the inner skin up the sides of the ship.

Another view of the question which has not been touched upon in the foregoing is the fact that even had the inner bottom of the "Arkansas" failed, the watertight subdivision undoubtedly would have saved the ship. While it is conceivable in that event that several of the larger compartments might have been flooded, there is every reasonable assurance that the vessel would have been safe.

Pittsburgh, Penn., in 1807

As of interest in showing the vast expansion of our cities during the past century, we publish the following statement of the "number of manufactories, artists and mechanicks" situated in Pittsburgh, Penn., in 1807. The matter is taken from "The Navigator: Containing Directions for Navigating the Monongahela, Alleghany, Ohio and Mississippi Rivers; with an Ample Account of These Much Admired Waters, from the Head of the Former to the Mouth of the Latter; and a Concise Description of Their Towns, Villages, Harbors, Settlements, Etc.," published by Zadok Cramer, Pittsburgh, 1808.

Pittsburgh is boosted very highly as may be seen from the accompanying extract, for which we are indebted to R. C. Hardman of Colorado Springs, Colo.

It is highly probable that Pittsburgh will become, from the many advantages which nature has placed within the grasp of its inhabitants—added to those which may arise from the addition of art—one of the most considerable inland manufacturing towns in the United States. Indeed we can draw at this time a tolerable good picture of our future eminence by a view of the present state and number of our manufactories, artists and mechanicks, each pursuing his business in a manner perhaps unparalleled, for vigilance, ingenuity and industry.

The following statement of the number of master workmen in each particular branch, with the manufactories, was taken in the fall of 1807: 1 cotton manufactory, having a mule of 120 threads, and one spinning Jenny of 40 threads, 4 looms, and a wool carding machine under the same roof; 1 glass works, on the opposite side of the Monongahela, for green, and one on this side for white glass, 2 breweries, 1 air furnace, 4 nail factories, one of which makes 100 tons of cut and hammered nails annually; 7 coppersmiths, tinplate workers and japanners; 1 wire weaving and riddle factory; 1 brass foundry, 6 saddlers and harness makers, 2 gunsmiths, 2 tobacconists, 1 bellmaker, 3 tallow chandlers, 1 brush maker, 1 trunk maker, 5 coopers, 13 weavers, 10 blue dyers, 1 comb maker, 7 cabinet makers, 1 turner, 6 bakers, 8 butchers, 2 barbers, 6 hatters, 4 physicians, 2 potteries of earthenware, 3 straw bonnet makers, 4 plane makers, 6 milliners, 12 mantua makers, 1 stocking weaver, 2 book binders, 4 house and sign painters, 2 portrait painters, 1 mattress maker, 3 wheelwrights, 5 watch and clock makers and silversmiths, 5 bricklayers, 5 plasterers, 3 stone cutters, 8 boat, barge and ship builders, 1 pump maker, 1 looking-glass maker, 1 lock maker, 7 tan yards, 2 rope walks, 1 spinning wheel maker, 17 blacksmiths, 1 machinist and whitesmith, 1 cutler and toolmaker, 32 house carpenters and joiners, 21 boot and shoe makers or cord wainers, 5 windsor chair makers, 13 taylors, 1 breeches maker and skin dresser, 12 school masters, 4 school mistresses, 33 taverns or public inns, 51 mercantile stores, 4 printing offices, 6 brickyards, 3 stone masons, 2 bookstores, 4 lumber yards, 1 maker of machinery for cotton and wool manufactories, one factory for clay smoking pipes, 1 indifferent copperplate printing press.

Neglected First Principles of Masonry Dam Design

By George Holmes Moore*

With ample provocation has Frederick P. Stearns declared that "It is a disgrace to the engineering profession that so many masonry dams have failed." And with the very soundest of premises has John R. Freeman predicted that all the text-books on dam design must be either entirely rewritten or else drastically revised.

Briefly stated, the indictment against the existing text-books and treatises may be expressed in the sweeping charge that they "amplify the unimportant and evade the essential." Such a charge demands evidence to sustain it; and to adduce such evidence is the purpose of the discussion which follows. Of the various neglected first principles of dam design, for brevity only a few of the more striking examples will be dealt with, under the following captions:

(1) The two-three triangle for gravity dams.
(2) The stress changes attendant upon overflow.
(3) Abortive analyses for ice thrust, bearing power and uplift.
(4) Professional attitude toward the arch and similar designs.

The Two-Three Triangle for Gravity Dams

In possibly no other branch of dam design is the amplification of the unessential so marked as here, for the "theoretic-profile" and "hyperbolic-curve" nonsense heaped upon what might be termed the standard gravity section is astounding indeed. Pages, chapters, even volumes are devoted to a discussion of gravity profiles which depart but negligibly from a simple basic section. Yet this basic section and the reasons for it are seldom more than touched upon in the haste of the authors to arrive at some pretentious profile of some more pretentious dam.

It is a fact, indeed it is the most fundamentally essential fact, that all gravity profiles, if superimposed upon each other, will produce a final "composite picture" (see Fig. 1.). The fact is not a new one, nor are the reasons far to seek. Yet where is the authority bold enough to declare that *all gravity profiles are merely variations of a simple triangle,* a triangle whose base dimension is two-thirds of its height? But one instance has come to the writer's attention in which the designer defied tradition and used the simple section unadorned. And this is the Roosevelt Dam, a structure which is fundamentally of the arch type.

The justification of the two-three triangle is simple indeed. Consider a triangular dam section as in Fig. 2. Let

h = depth below water level;
b = base-length;
w = weight of masonry in lb. per cu.ft.;
62.5 = weight of water in lb. per cu.ft.;
k = a constant, being the ratio of b to h necessary by the middle-third criterion.

By simple hydraulics, the overturning moment of the water is:

$$M_o = 62.5 \frac{h^2}{2} \times \frac{h}{3} = 62.5 \frac{h^3}{6}$$

Similarly the resisting moment of the masonry is:

$$M_r = w \frac{b \times h}{2} \times \tfrac{2}{3} b = \frac{w b^2 h}{3} = \frac{w k^2 h^3}{3}$$

The middle-third requirement involves (for solid dams) a minimum safety-factor of 2 against overturning about the toe as a pivot, i.e.,

$$M_r = 2 M_o$$

or,

$$\frac{w k^2 h^3}{3} = 2 \left(\frac{62.5\, h^3}{6} \right)$$

whence

$$k = \sqrt{\frac{62.5}{w}}$$

This is an equation which enables us to determine the ratio of section base to section height for any density of material. Thus:

For w = 160 lb. per cu.ft., k = 0.625, whence b = kh = 0.625 h
For w = 150 lb. per cu.ft., k = 0.646, whence b = kh = 0.646 h
For w = 140 lb. per cu.ft., k = 0.668, whence b = kh = 0.668 h

These ratios amply justify a typical section whose base is two-thirds its height, since the construction material for any impounding structure rarely or never weighs less than 140 lbs. per cu.ft.

This, then is the simple basis for the inevitably recurring two-three triangle which is approximated in nearly all standard sections for gravity dams.

The above analysis is not claimed to be new. The point and protest intended to be made is that none, or almost none, of the authorities present it as vitally essential. And, on the other hand, all (or almost all) authorities dwell upon the various profiles as distinct designs rather than as comparatively unimportant variations of a central theme.

Stress Changes Attendant upon Overflow

Of all the essential considerations of dam design, the analysis of the stress-changes which result as the water-level of a reservoir rises to and above the crest level of the impounding dam is possibly the one most sadly neglected. In fact, few, even among the designers, seem to appreciate the importance of such an analysis.

We know what section is safe from overturning when the water-level is at its crest. But where is the authority who will tell us how soon our safety-factor drops from the 2 of so called safety to the 1 of imminent disaster? Where is the standard textbook which warns the novice that a "safe" section for a 30-ft. head becomes a menacing one if topped by a 10-ft. flood? Yet these are facts, as will be shown.

A little trade booklet is issued under the striking title of "The Dams That Failed." Here we find a collection of photographs and other data concerning failures of all kinds and classes. An inspection of this discloses at least one absorbingly interesting fact, namely that *no dam over 80 ft. in height ever has failed.* And further it appears that, of the masonry structures which have given way, by far the greater number have been of the *overfall* type; i.e., low dams subject to considerable depths of overflow. This seems something more than coincidence.

Refer to the section sketch Fig. 3. Using the method previously adopted, with some slight variations, let:

P = overturning resultant of water pressure;
h_1 = height of section, or water depth for crest-line;

*City Engineer's Office, Seattle, Wash.

h_2 = water depth for failure-level;
$x = h_2 - h_1$ = depth of overtopping flood;
y = depth of water resultant below water level;
$z = h_2 - y$ = vertical distance between water-resultant and base-level;

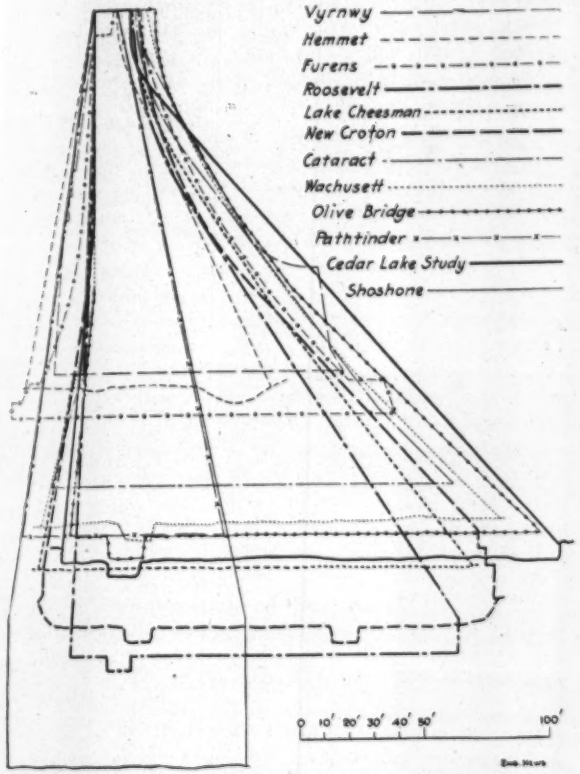


FIG. 1. SUPERIMPOSED PROFILES OF SOME WELL KNOWN MASONRY DAMS

("Cedar Lake Study" is a preliminary or study section for the Cedar Lake Dam of the Seattle water-supply; it is not the section finally adopted.)

and M_{of} = overturning moment for failure conditions.

The following relations may be deduced from Fig. 3:

$$M_{of} = P\,(h_2 - y)$$

$$P = \frac{62.5}{2}\,(h_2^2 - x^2)$$

$$y = \tfrac{2}{3}\left(\frac{h_2^3 - x^3}{h_2^2 - x^2}\right)$$

from which, after simplifying, we obtain

$$M_{of} = \frac{62.5}{6}\,(h_2^3 - 3\,h_2 x^2 + 2\,x^3)$$

At "failure-level" (water-depth = h_2), the resisting moment will be just balanced by the overturning moment; or

$$M_{of} = M_r = 2\,M_o = 2 \times \frac{62.5\,h_1^3}{6}$$

and we have

$$\frac{62.5}{6}\,(h_2^3 - 3\,h_2 x^2 + 2\,x^3) = 2 \times \frac{62.5\,h_1^3}{6}$$

Substituting for x its value of $h_2 - h_1$, we arrive at a final value of

$$h_2 = \frac{4}{3}\,h_1$$

which demonstrates the proposition as previously stated, that a safe section for a 30-ft. head becomes a menace or a failure if topped by a 10-ft. flood. And this without considering the accessory agents of vacuum, tremor and flotational effects of submergence. Small wonder that so many overfall dams are among the list of failures!

The above discussion brings out only the limiting condition of overflow depth, viz., that which will produce failures by overturning. But the variation of stresses for intermediate water levels will be seen in Figs. 4, 5 and 6.

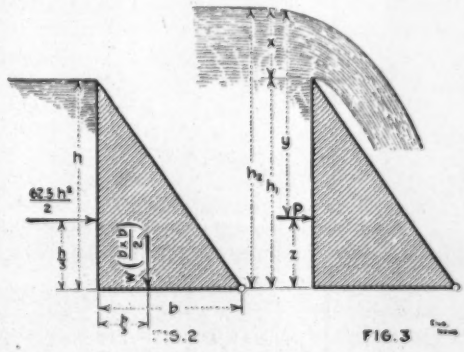
From an inspection of Fig. 4, it will be evident that the 66-ft. Austin (Tex.) dam was already dangerously stressed by the 11-ft. overflow, even without the accessory factors of municipal neglect and of that fatal hole beneath the downstream toe which was discovered by the historic observer armed with a fish-pole. Indeed, as we read of the parallel current which cut with destructive force along the toe of the doomed structure, we wonder not at all that it failed but only that it stood so long.

Lest the foregoing be assailed on the ground that this dam failed by sliding and not by overturning, the writer wishes here to emphasize his view that a dam may fail essentially by overturning and yet be found upright after failure. For it is in the initial fracture of the base-bond, in the formation of the relatively small openings which instantly permit the simultaneous action of hydraulic uplift and of hydraulic lubrication, that the writer believes the genesis of most dam failures to lie.

This view should also be borne in mind in the consideration of the section dealing with uplift.

ABORTIVE ANALYSES FOR ICE THRUST, BEARING POWER AND UPLIFT

ICE THRUST—With regard to ice thrust it may be well to note that most authorities pass over it as something indeterminate. Ice thrust is merely stated to be a factor well worthy the attention of the designer, and the latter, though carefully coached and cozened in other lines, is here turned adrift and left to the by-no-means-tender mercy of the falling thermometer.*



However, there is at least one classic instance in which ice thrust seems to have actually been figured and analyzed. This was in the instance—also classic—of the

*Perhaps Mr. Moore has overlooked the 80 pages on "Provision for Uplift and Ice Pressure in Designing Masonry Dams," by the late C. L. Harrison, and 20 other engineers in Trans. Am. Soc. C. E., Vol. LXXV, pp. 143-225. 1912.—Ed.

**By no means. Here again the point and protest is that so much of value is allowed to lie buried in the records of the various technical societies. For if dams were designed only by experienced engineers, and by them only after exhaustive research, the roll of disasters would be far shorter than it is in fact.—Author.

design of the Quaker Bridge Dam (see report of The Board of Experts, Wegmann, p. 158). Here we find the somewhat mouth-filling quantity of a stress of "43,000 lb. per lin.ft. at the highest ice line." How the eminent gentlemen who contributed this lone bit of wisdom arrived at the figure of 43,000 lb. is a mystery indeed. Possibly their mental attitude was that of the sportsman who, having sworn to ninety-nine dead quail to one shot, disdained to "risk his veracity for one more bird." This at least would explain why a figure of 45,000 lb. or even 50,000 lb. was not submitted as being amply provided against, for the truth of the matter is that one is as sensible as the other, and neither seriously menaces the "safety factor of two." For, even considering a force of 50,000 lb. per lin.ft., and this applied at the crest of a

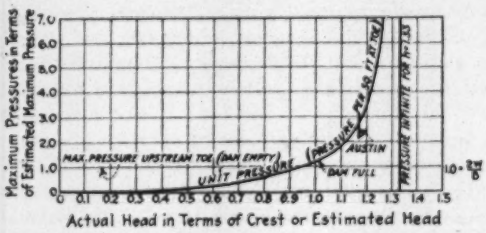

Maximum Pressures in Terms of Estimated Maximum Pressure
MAX. PRESSURE UPSTREAM TOE (DAM EMPTY)
UNIT PRESSURE
AUSTIN
DAM FULL
PRESSURE INFINITE FOR H=1.33
Actual Head in Terms of Crest or Estimated Head

Fig. 4. Variation of Pressure-Intensity under Toe of Dam with Depth of Water

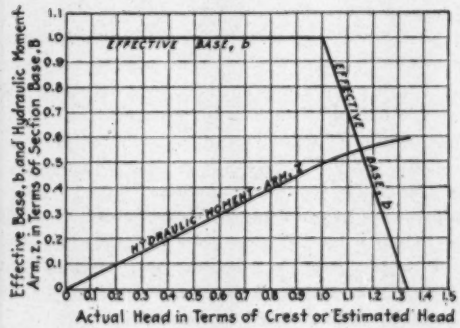

Effective Base, b, and Hydraulic Moment-Arm, z, in Terms of Section Base, B
EFFECTIVE BASE, b
HYDRAULIC MOMENT-ARM, z
Actual Head in Terms of Crest or Estimated Head

Fig. 5. Variation of Effective Base and Hydraulic Moment Arm with Head of Water

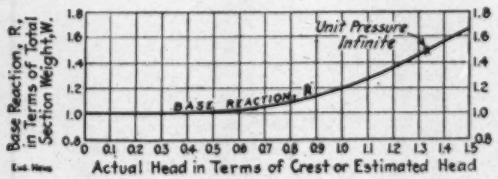

Base Reaction, R, in Terms of Total Section Weight, W.
Unit Pressure Infinite
BASE REACTION, R
Actual Head in Terms of Crest or Estimated Head

Fig. 6. Variation of Total Base Reaction with Head of Water

section 258 ft. high, the overturning moment due to ice thrust would be only one-fourteenth of the overturning moment due to the water pressure for water-level at crest. For, the moment of ice thrust would be

$$M_i = 50{,}000 \times 258 = 12{,}900{,}000 \ ft.\text{-}lb.$$

while the moment of hydraulic force would be

$$M_w = \frac{62.5}{6} \times 258^3 = 178{,}900{,}000 \ ft.\text{-}lb.$$

The above computation, even though it adds nothing to justify any particular limiting stress, certainly is of value in showing the relation between the stress assumed and the moment of hydraulic force. And this relation is not found in the textbooks which have so far come to the writer's attention.

Before leaving this subject it may be well to point out that the dangers of ice thrust are by no means confined even in still ponds or lakes. For, if the ice-sheet cracks, spreads apart and freezes again, indeterminate forces of enormous value may result; and it is quite possible that under such conditions the destructive force imposed upon a dam may be limited only by the ultimate compressive resistance of the ice itself.

Bearing Power—The standard presentation of the subject of the bearing power of dam foundations is another method which seems to seek less to teach than to mislead. Stripped to its essentials, the bearing power of the foundation rock is presented as being only required to sustain a column of rock or other dam material, equal in height to the maximum height of the finished dam. Thus presented the subject seems satisfactorily simple, but any one who has studied the problems of dam failure knows that dams do not fail by any such simple direct compression. And the efforts to justify this line of reasoning have resulted in one of the most striking absurdities of present practice in dam design, viz., the specification of a compressive stress which gives a safety-factor of 10 to 100 for a structure whose safety-factor against overturning is only slightly more than 2.

As a matter of fact, a better method of treatment would be to follow the variation of base pressure as shown in Fig. 4, which gives the progressive increase of this unit up to the value of infinity, reached at the limiting value of overflow-head discussed previously in this article.

Turning now to the curve itself (which, to be thoroughly understood, needs to be considered in connection with the curves for effective base length, total base reaction and hydraulic moment-arm as shown in Figs. 5 and 6), we find that the unit-pressure at the toe varies slowly and steadily from the condition of reservoir empty to the condition of water-level at dam crest. For the latter condition, the force resultant just cuts the downstream edge of the middle third, and the safety factor is exactly 2. As the water-level rises above the crest elevation, the point of application of the resultant moves downstream of the middle third, causing the effective base length to decrease rapidly, with a resultant rapid increase in the intensity of unit-stress at the downstream toe. The latter rises sharply to infinity as the overflow-head rises to the limiting or failure-level previously discussed; and under these conditions crushing and destruction must inevitably occur regardless of the strength of the dam material or foundation composition.

Since the above curve is actually a graphic exposition of the manner in which the compressive stress rises to destructive proportions, it seems rather farcical to assume empirical limiting "Maximum Pressures" such as 14.65 or 15.16 tons per sq.ft. (Wegmann, 6th ed., p. 40). Especially is this true since the ultimate compressive strength of stone varies from 140 to 1400 tons per sq.ft., and even ordinarily good concrete should show an ultimate of 140 tons per sq.ft.

Viewed from some angles, the subject of bearing power or foundation-strength analysis seems open to at least certain slight improvements.

3. Uplift—The analyses for uplift as customarily pre-

quoted leave much to be desired. Indeed there seems ground for a suspicion that they have been developed by the dubious method of working backward from a known answer. Or if this charge is groundless, what defense can be advanced for the most common assumption, namely: that the uplift varies from a maximum at the heel to zero at the toe, and further that this maximum may be taken as two-thirds of the unit-pressure due to the full hydraulic head (see Morrison and Brodie, "High Masonry Dam Design," p. 15)?

Both parts of this assumption are vulnerable to attack. As far as the uniform variation from a maximum to zero, the origin of the assumption is easily seen to lie in the treatment of groundwaters, and especially in the laws of flow through a porous material of uniform composition. For earth dams, the method is undoubtedly defensible. But for rock foundations, it can be nothing more than an empirical assumption, based upon neither theory nor experiment, but really only a first halting step toward the vigorous treatment that must finally be pursued.

Having once admitted that the design must allow for the pressures which inevitably follow the entrance of water beneath the base of a dam, it is difficult to defend an assumption of decrease to zero at the toe, and especially difficult to accept the idea that the maximum intensity shall be only two-thirds of the amount known to be most probable.

This is especially true since we have had sinister light on the lesson that uplift is most to be expected in a seamy or stratified material. And under these conditions, not only is it most probable that the maximum intensity will be the full amount due to total hydraulic head, but also it is entirely possible that, by reason of a closed exit to the assumed under-dam crevice, this full amount will continue undiminished from heel to toe of the section.

These conditions may be stated as follows: We must assume either (1) uplift due to full head at heel varying uniformly to zero at the toe, or (2) uplift due to full head acting undiminished from heel to toe.

This latter especially is, as the eminent Sir Raleigh once remarked, "sharp medicine indeed, but sure cure for all diseases" likely to result from uplift.

The application of the two assumptions to the fundamental dam triangle is discussed below:

A. UNIFORMLY VARYING UPLIFT

Assume a triangular dam designed, disregarding uplift pressure on base, having base b; height h, and weight of one foot section W. The resisting moment is ⅔ Wb. To find the new base, X, for which the resultant resisting moment, found by deducting uplift moment from weight moment, equals the original moment as above found.

The required relation is given by the equation:

$$\tfrac{2}{3}\, W_x X - \tfrac{1}{3}\, 62.5\, hX^2 = \tfrac{2}{3}\, Wb$$

in which the second term is the moment of the uplift, and uplift pressure is assumed to vary from that due to the full head h at the upstream toe, to zero at the downstream toe.

Now, as both the original and the modified sections are triangular, and of the same height h, their weights are in the same ratio as their base lengths, or

$$W_x = \frac{X}{b}\, W$$

Also, for dam material of specific gravity 2½ (about 156 lb. per cu.ft.) the weight of the original section is

$$W = \tfrac{1}{2}\, bh \times 2\tfrac{1}{2} \times 62.5.$$

Substituting these values in the above equation gives the result

$$X = \sqrt{\tfrac{2}{3}}\, b = 1.291\, b$$

B. FULL UNIFORM UPLIFT

Assume dam designed, disregarding uplift pressure on base, having base b; height h, and weight of one foot section W. The resisting moment is then ⅔ Wb. To find the new base X, for which the resultant resisting moment, found by deducting uplift moment from weight moment, equals the original moment as above found.

The required relation is given by the equation:

$$\tfrac{2}{3}\, W_x X - \tfrac{1}{2}\, 62.5\, h X^2 = \tfrac{2}{3}\, Wb.$$

in which the second term represents the moment of uplift due to full hydrostatic head acting over the entire base. As in the preceding case, we have

$$W_x = \frac{W}{b}\, X$$

and

$$W = \tfrac{1}{2}\, bh \times 2\tfrac{1}{2} \times 62.5.$$

Substituting these gives

$$X = \sqrt{\tfrac{2}{3}}\, b = 1.583\, b$$

This means that if the original base was 100 ft., the new base X is 158.3 ft.

If it be doubted that the foregoing analyses, and the sections resulting from them, are an index of the coming standards of gravity-dam design, an inspection of all recent sections of high-gravity dams will reveal the slow change to more and more ample sections. And, furthermore, the writer believes that at least one German designer has already put forward a section which satisfies the requirements of the first and less drastic of the treatments just given. Finally, he who reads may reckon that there is food for thought in the serious consideration given to types of dams which are as far removed as possible from dependence upon simple gravity.

Professional Attitude toward the Arch and Similar Designs

Just why the discussions in our scientific societies and journals should contain so much faint praise, or eager condemnation, of the arch type of dam, is probably more of a question for an accountant or a philosophic pessimist than for an engineer.

The fact is noted in Engineering News, Vol. 64, p. 447, *"No failure of any such dam has ever been recorded"*! ! ! No arch dam of *any* size, shape or parentage has *ever* failed! Surely this should be sufficient and final.

And yet, turning to p. 419 of Wegmann's sixth edition, we find once again that "Mr. Wade is of the opinion, which we share, that gravity dams should be built straight in plan, as the employment of a curved plan does not give sufficient advantage to warrant the increase in the volume of the masonry which it involves. The use of curved dams, designed as those demonstrated above is therefore restricted to comparatively narrow valleys."

And this in the name of engineering *science!*

Final Word

It is time for change—yes, long past time for us to face squarely and openly certain inherent weaknesses of design, weaknesses long known to exist by those in the profession, but seldom or never honestly admitted to the public.

Either in the name of progress, or else in the shame of recurrent failure, let us fling away childish things and sturdily press forward to the goal of The Dams That Do *Not* Fail.

⁂

Anthracite Coal Shipments in the first seven months of 1913 were 40,339,706 tons, against 40,113,648 tons in the record year of 1911. The total production for the latter year was about 70,000,000 tons.

Precise Leveling in New York City*

By Frederick W. Koop†

SYNOPSIS—The five boroughs comprising New York City and different engineering departments and bureaus in each of the boroughs have long used benchmarks and datum planes of their own. This, of course, has resulted in some confusion, and an immense amount of office work in adjusting grades for engineering-construction work. The great advantage of having a really precise and comprehensive level net was pointed out by the society The Municipal Engineers of the City of New York, and through its efforts an appropriation for the work was obtained. This paper gives in detail the organization and methods of the work, the method of computation, of reciprocal leveling for river crossings, and the whole procedure in both field and office operations in more complete and useful form than, we believe, any such work has hitherto been described.

On September 23, 1908, a Committee on the Unification of Datum Planes consisting of seven engineers representing all boroughs, was appointed by the President of The Municipal Engineers of the City of New York. This Committee was to investigate the desirability of the establishment of a uniform system of benchmarks throughout New York City, and to recommend, if possible, the adoption of a suitable datum plane which could be used by all of the city departments.

The urgent need of such action had become more and more evident to the city's engineers, who had realized for a long time that there was a deplorable lack of reliable benchmarks, and that the establishment of a uniform system of standard benches that could be used in common by all departments was greatly to be desired.

After a careful investigation and report by this Committee, The Municipal Engineers of the City of New York adopted resolutions requesting the Board of Estimate and Apportionment to establish a system of benchmarks throughout the city, all referring to mean sea level at Sandy Hook; such benchmarks to be of permanent construction and so placed as to be convenient for use by all the Boroughs and Departments. It was further recommended that a list of such benchmarks be published after adoption.

Cost of Work—On Feb. 19, 1909, the Board of Estimate and Apportionment adopted a resolution appropriating $10,000 for the running of precise levels and the establishment of benchmarks in connection with the topographical work of al boroughs, and on Mar. 2 this resolution was approved by the Board of Aldermen. During February, 1911, an additional $5000 to complete this work was voted, making a total appropriation of $15,000.

From this sum were paid the salaries of the men employed on the work, office rentals, cost of instruments, railroad fares and incidentals; in fact, everything in connection with the work excepting the salary of the author who was Assistant Engineer in charge, which was paid by the Board of Estimate and Apportionment.

Plan of Work—Briefly stated, the plan proposed was as follows:

(1) To call on the United States Coast and Geodetic Survey for a list of benchmarks in and around New York City, with their locations and elevations. These benches had all been adjusted, referred to mean sea level at Sandy Hook, and though few in number, were to form the basis of future leveling in New York City and vicinity.

(2) To make a reconnaissance of each borough, and select the level lines necessary to give a good level net in each: also, to locate suitable points for benchmarks along these lines and to establish others so that there would be at least one permanent benchmark to each mile of level line.

(3) To call on the topographical bureaus and other engineering departments of the city for a list of benchmarks along the proposed level lines, with the datum planes to which they were referred and the history concerning the adoption of such planes, and to ascertain if possible what other departments or organizations made a practice of referring their elevations to these datum planes.

(4) To adopt the methods now in use by the Coast Survey both in the field operations and in the reduction of the notes and office computations, in order to secure the best results for the city and to attain a degree of precision in the leveling at least equal to that attained by the Coast Survey in their precise leveling during recent years.

Instruments†—An order was placed with the Bausch & Lomb Optical Co., of Rochester, N. Y., for two precise levels with tripods and three metric leveling rods with foot plates and pins, all of which were to be of the latest Coast Survey type* and to be made from specifications furnished by the instrument division of the Coast Survey.

The rods ordered were the standard Coast Survey metric rods. All differences of elevation are therefore in meters and the horizontal distances in kilometers. When published in final form the elevations will also be given in feet.

The metric system was found, as usual, to be most convenient and practical, and on account of its adoption by the national surveys of the United States and of European countries, we were able to use in our computations many of the tables and formulas devised by these surveys.

Recovery of Old Coast Survey Benchmarks—During 1886-'87 the United States Coast Survey established a series of benchmarks along a portion of the waterfront of New York City, referring their elevations to mean sea level as determined by tidal observations made at Sandy Hook during 1876-81. While awaiting the completion of the instrumental outfit, a search was begun to recover as many of these benchmarks as possible. Comparatively few were found however, the majority of them having been destroyed.

Reconnaissance—A reconnaissance was then made over Brooklyn, Queens and Richmond Boroughs, level routes were selected and a level net planned for each. These level lines included all the Coast Survey benchmarks recovered and the benchmarks of the various city departments, about one to each mile. A reconnaissance of the boroughs of Manhattan and the Bronx was made during the following winter.

Inspection of Instruments—In June, 1909, the precise levels, rods, etc., were delivered by the makers to the instrument division of the Coast Survey for inspection, such inspection and approval being a condition of the contract. The instruments and rods were tested as to material and workmanship in the author's presence, and the leveling rods were then sent to the United States Bureau of Standards to have their absolute lengths determined.

The instruments and rods were approved and passed by the Coast Survey, but the tripods, foot plates and foot pins were returned to the maker for alteration. The outfit was finally delivered complete in September, 1909.

Field Party—A field party was then organized, consisting of the Assistant Engineer in charge, who acted as observer, three rodmen, one of whom acted as recorder, and one man whose duty it was to shade the instrument from the sun with a large umbrella and to prepare benchmarks. This made a total of five men in the party, while Coast Survey leveling parties consist of six men.

The levels and rods were tested in the field and on Oct. 13, 1909, the actual running of the level lines was commenced. Except during the winter months when little precise leveling could be done, this work continued uninterruptedly until May 1, 1912, when the field party was disbanded. All of the fieldwork, covering each of the boroughs with primary level lines and connecting one with the other, was accomplished with this one level party of five men, which very often was reduced to four.

PRELIMINARY FIELD OPERATIONS

The Determination of C—The Coast Survey type of precise level being very simple in construction requires but one adjustment in the field and that adjustment should be examined daily. This is accomplished by a modification of the well known peg method and is called the "determination of C," or the error of the level.

The numerical value obtained for C gives a control of the non-parallelism of the axis of the bubble and the line of sight and enables proper corrections to be applied, when

*Abstract of a paper read before The Municipal Engineers of the City of New York, May 28, 1913.

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†Further information on turning points used, type of targets used for river crossings and detailed description of other instruments will be published in a special report of the Board of Estimate and Apportionment.

*Described in "Engineering News," Mar. 23, 1911, p. 356; July 2, 1903, p. 2.

TABLE I. DETERMINATION OF THE INSTRUMENTAL CORRECTION C.

Date: Sept. 28, 1911. Forward-Backward

Sta. C. No. of station	Thread reading backsight	Mean	Thread intervals	Sum of intervals
	1807		14	
I	1821	1821.0	14	
	1835		28	28
	1448		14	
N	1462	1462.0	14	
	1476		28	56
		+ 3283.0		
		— 3283.4		
		346) —0.4	C = —0.001	

From B. M. Wind: C. Rod and temp.	To B. M. Thread reading foresight	Mean	Thread intervals	9:00 A.M. Sum of intervals
	0795		100	
	0895	0895.0	100	
	0995		200	200
	2288		101	
	2389	2389.0	101	
	2490		202	402
		3284.0		—56
Correction for curv. and ref.		—0.6		346
		3283.4		

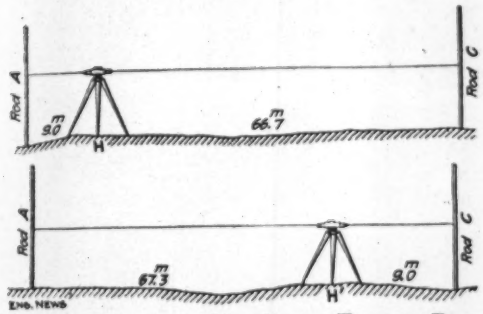

FIG. 1. ILLUSTRATING METHOD OF TESTING PRECISE LEVEL

necesary, to results obtained when the line of sight is inclined to the vertical.

It was considered a better policy on this work to test the instrument every morning and thus be sure of its adjustment, than to test it at the end of the day's leveling and find possibly that a correction would have to be applied to the day's work. The determination of C as actually made in the field is illustrated in Fig. 1 and Table I.

The value of C is expressed as follows:

$$C = \frac{(\text{sum of near rod readings}) - (\text{sum of distant rod readings})}{(\text{sum of distant rod intervals}) - (\text{sum of near rod intervals})}$$

The instrument is set up at H, as shown in the upper illustration, Fig. 1, about 9 m. from rod A and about 66.7 m. from rod C. These distances are not measured, but should be approximately as indicated. A reading is taken on rod A and is placed in the backsight column, the mean reading in this instance being 1821.0. A reading is next taken on rod C and is placed in the foresight column, the mean being 0895.0. All three wires are read and recorded each time.

The instrument is then moved to station H' (lower figure), the rods remaining as before and the horizontal distances being about in the reverse order. A reading is taken on rod C and placed in the backsight column, the mean being 1462.0. The two readings in the backsight column are called the "near rod readings." A reading is then taken on rod A and placed in the foresight column, the mean being 2389.0. The two readings in the foresight column are called the "distant rod readings."

The sum of the distant rod readings is then taken and a correction of —0.6 mm. is applied for curvature and refraction, the two corrections for the two distant rods being combined. This corrected sum of the distant rod readings is then substracted from the sum of the near rod readings, giving in this case a residual of —0.4. This residual is divided by the sum of the distant rod intervals minus the sum of the near rod intervals, which in this case is 346, and a quotient of —0.001 is obtained, which is the value of C. The rod readings must be taken with the bubble in the middle of its tube.

When C is less than 0.005 no adjustment of the instrument is necessary, and in fact it is not of much importance until C becomes equal to 0.01 or more. When the instrument must be adjusted, it is done by raising or lowering one end of the level vial and not by moving the reticule.

Adjustment of the Level—The adjustment is made as follows: Point to the distant rod with the bubble in the middle of its tube and read the three horizontal wires. Move the telescope so as to raise the middle wire by an amount equal to C times the rod interval. While holding the telescope in this position bring the bubble to the middle of the tube by raising or lowering one end of the level vial. If C is negative the middle hair must be lowered on the rod and if plus it must be raised. After adjusting the instrument, a new determination of C must be made. The cross-hairs must never be disturbed as these have been permanently adjusted for collimation by the instrument maker.

A very high grade of workmanship was used in the construction of these instruments as is testified to by the constancy with which they retained their adjustment. Level No. 1, being used the most, was tested every day, but required adjusting only about once in three weeks. Level No. 2 was frequently used for months at a time and required adjusting about once in three months, and at no time was the angle between the line of collimation and the tangent to the level vial at its middle point greater than 4".

GENERAL INSTRUCTIONS FOR PRECISE LEVELING*

(1) Except when specific instructions are given to proceed otherwise, all lines are to be leveled independently in both the forward and the backward direction.

(2) The line of levels is to be broken up by benchmarks into sections from 1 to 2 km. long, except where special conditions make shorter sections advisable.

(3) All old benchmarks are to be called by their old names and are to be fully described by quoting the old description, if one is available, and by making additions to it.

(4) It is desirable that the backward measurement on each section should be made under different atmospheric conditions from those which occurred on the forward measurement. It is especially desirable to make the backward measurement in the afternoon if the forward measurement was made in the forenoon, and vice versa. The observer is to secure as much difference of conditions between the forward and backward measurements as is possible without materially delaying the work for that purpose.

(5) On all sections upon which the forward and backward measures differ by more than 4.0 mm. $\sqrt{K}$ (in which K is the distance leveled between adjacent benchmarks in kilometers), both the forward and backward measures are to be repeated until two such measures fall within the limit.

(6) Whenever a blunder, such as a misreading of 1 dm. or of 1 m., or an interchange of sights (the backsight being recorded as a foresight) is discovered in any measure after its completion, and the necessary correction applied, such measure may be retained, provided there are at least two other measures over the same section which are not subject to any such uncertainty.

(7) The programme of observation at each station is to be as follows: Set up and level the instrument. Read the three lines of the diaphragm as seen projected against the front (or rear) rod, each reading being taken to the nearest millimeter (estimated), the level bubble being kept continuously in the middle of the tube. As soon as possible thereafter read the three lines of the diaphragm, as seen projected against the rear (or front) rod, estimating to millimeters as before, and keeping the bubble continuously in the middle of the tube.

(8) At each rod station the rod thermometer is to be read to the nearest centigrade degree and the temperature recorded.

(9) At stations of odd numbers the backsight is to be taken before the foresight, and at even stations the foresight is to be taken before the backsight.

(10) The maximum difference in length between a foresight and the corresponding backsight is to be 10 m. The actual difference is to be made as small on each pair of sights as is feasible by the use of good judgment without any expenditure of time for this particular purpose.

(11) The recorder shall keep a record of the rod intervals subtended by the extreme lines of the diaphragm on each backsight, together with their continuous sum between benchmarks. A similar record shall be kept for the foresight. The two continuous sums shall be kept as nearly equal as is feasible, without the expenditure of extra time for that purpose, by setting the instrument beyond (or short of) the middle point between the back and front rods. The two continuous sums shall not be allowed to differ by more than a quantity coresponding to a distance of 20 m.

(12) Once during each day of observation the error of the level should be determined in the regular course of the leveling and recorded in a separate opening of the record book, as follows: The ordinary observations at an instrument station being completed, transcribe the last foresight reading as part of the error determination, call up the back rodman and have the rod placed about 10 m. back of the instrument; read the rod; move the instrument to a position about 10 m. behind the front rod; read the front rod, and then the back rod. The rod readings must be taken with the bubble in the middle of the tube. The required constant C is determined as previously explained. The total correction for curvature and refraction must be applied to the sum of the distant rod readings before using it in this formula. The level should not be adjusted if C is less than

*These instructions are in general practically identical with those prepared by John F. Hayford, for the precise level work of the United States Coast and Geodetic Survey; see Johnson's "Theory and Practice of Surveying," 17th Edition, p. 703. Additions and variations have been made by Mr. Koop to particularly adapt them to city work.

0.005. If C is between 0.005 and 0.010 the observer is advised not to adjust the level, but if C exceeds 0.010 the adjustment must be made. If a new adjustment of the level is made, C should at once be redetermined. It is desirable to have the determination of level error made under the ordinary conditions as to length of sight, character of ground, elevation of line of sight above ground, etc. The level must be adjusted by moving the level vial, not by moving the reticule.

(13) Notes for future use in studying leveling errors shall be inserted in the record, indicating the time of beginning and ending of the work for each section, the weather conditions especially as to cloudiness and wind, and whether each section of the line is run toward or away from the sun; and such other notes as may be of value in studying errors.

(14) The instrument shall be shaded from the direct rays of the sun, both during the observations and the movement from station to station.

(15) The maximum length of sight shall be 150 m., and the maximum is to be attained only under the most favorable circumstances.

(16) At the beginning and end of the season and at least twice each month during the progress of the leveling, the three-meter interval between metallic plugs on the face of each level rod shall be measured carefully with a steel tape, which shall be continuously kept in the party throughout the season for that purpose. The rod temperature at the time of each of these measures must be recorded. The purpose of these measures is to detect changes in the length of the rods rather than to determine the absolute lengths. The absolute lengths are determined at the office between field seasons. At least once each month, during the progress of the leveling, the adjustment of the rod levels shall be tested and a statement inserted in the record showing the manner in which the test was made, whether the error was found to be without the limit stated below, and whether an adjustment was made. The test must determine the inclination of the rod to the vertical measured in the plane of sight from the instrument to the rod, as well as at right angles to that plane, when the bubble of the rod is held at the center. If the deviation from the vertical exceeds 10 mm. on a three-meter length of the rod, the rod level must be adjusted.

Field Notes—Table II shows the form of record for the field notes. The first column contains the number of the stations, the second the reading of each horizontal wire for the backsight, the third the mean of the three readings, the fourth the stadia intervals, and the fifth the sum of the stadia intervals. The first column on the right-hand page contains the rod and temperature C° for each foresight, the second the reading of each horizontal wire for the foresight, the third the mean, the fourth the stadia interval, and the fifth the sum of the stadia intervals.

The unit in the elevation record is the millimeter. The instrument stations are numbered consecutively from benchmark to benchmark, this particular run being from B. M. No. 3 at 141 Broadway to B. M. No. 4 at the General Post Office. Such portions of the computation as are shown as forming a part of the record are kept up by the recorder as the work progresses.

CORRECTIONS IN PRECISE LEVEL WORK

Precise leveling differs from ordinary leveling not only in the type of instrument used and in the method of observation, but also in the application of certain corrections to the field results. In no branch of surveying is a knowledge of the sources of errors and of the methods of eliminating the same more important. However, with the present type of instrument and the present method of taking the observations, precise spirit leveling has become a very simple matter and the corrections applied are few, but very important. The largest and most important corrections applied to the field results are the rod length and rod temperature corrections.

Rod Length Correction—The length correction is the amount the rod differs from the length which its graduations indicate, and its application rectifies the results of read rod readings. In applying the length correction, the mean length of the two rods at 0° C. is determined and the excess or deficiency per meter of length is applied to each of the various differences of elevation determined by the leveling.

The correction is equal to the difference of elevation in meters times the excess or deficiency per meter. It is plus when the mean rod is too long at 0° C. and minus when it is too short. Any other temperature may be used as a standard. The following illustration will make the reason for this correction more clear, the principles being applicable also to ordinary leveling.

In Fig. 2 (small Fig. 1) the line of sight AB is exactly 3 m. above the foot of the rod D, and the absolute length of

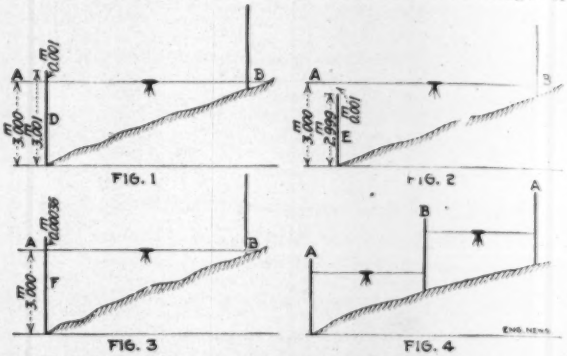

FIG. 2. ILLUSTRATING METHOD OF APPLYING ROD AND TEMPERATURE CORRECTIONS

the rod at 0° C. is 3.001 m. The rod being longer than its graduations indicate, the 3-m. mark is above the line of sight and the actual reading is less than it should be.

The difference of elevation is therefore in error by the amount that the rod is long, which in this case is 1 mm., or 0.333 mm. per meter of difference of elevation; this is the length correction, and it will have the same sign as the measured difference of elevation.

In Fig. 2 (small Fig. 2) the rod E is 1 mm. short at 0° C., and the reading will therefore be too great by that amount. In this case, the length correction will have the opposite sign to the measured difference of elevation.

If these particular rods were used together their mean length at 0° C. would be taken, which in this case is exactly 3.0 m. The length correction would then be zero. Usually, however, leveling rods are longer than their graduations indicate, and the length corrections are very often as much as +0.5 mm. per meter of difference of elevation.

The length correction may vary from year to year on account of changes taking place in the rods. The experience of the Coast Survey shows that paraffined rods increase slightly in length during the first season and preserve nearly

TABLE II. FORM OF FIELD NOTES: NEW YORK CITY PRECISE LEVELS

Date: Dec. 10, 1911 Forward—~~Backward~~
Sun: C.

No. of station	Thread reading backsight	Mean	Thread intervals	Sum of intervals
1	1124		66	
B.M. 3 at 141	1190	1190.3	67	
Broadway	1257		133	133
2	1386		73	
	1459	1459.0	73	
	1532		146	279
3	1865		65	
	1931	1929.7	64	
	1994		129	408
4	1902		53	
	1955	1955.3	54	
	2009		107	515
5	1730		60	
	1790	1790.0	60	
	1850		120	635
6	1833		28	
	1861	1861.3	29	
	1890		57	692
7	1616		17	
	1633	1633.3	18	
	1651		35	727
		11818.9		
		+683.6		

From B.M. : 3 To B.M. : 4
Wind: C. 10 A.M.

Rod and temp.	Thread reading foresight	Mean	Thread intervals	Sum of intervals	
A	1984		63		
9	2047	2047.0	63		
	2110		126	126	
C	1329		76		
10	1405	1405.0	76		
	1481		152	278	
A	1626		64		
9	1690	1690.3	65		
	1755		129	407	
C	1510		52		
9	1562	1562.0	52		
	1614		104	513	
A	1240		58		
9	1298	1298.0	58		
	1356		116	629	
C	1646		25		
9	1671	1671.3	26		
	1697		51	680	
A	1445		17	B.M. 4 at N.W.	
9	1462	1461.7	16	cor. of G. P. O.	
	1478		33		
		11135.3		713	

a constant length thereafter. This was also true of the Board of Estimate rods.

Rod Temperature Correction—The material of which the the rods are made has a coefficient of expansion of 0.000004 per degree centigrade. For any other temperature than that at which the rod is standard a correction must be applied to reduce observed values to true values. The temperature correction therefore depends upon the coefficient of linear expansion of the rods, the temperature at the time of observation and the difference of elevation.

Fig. 2 (small Fig. 3) illustrates this principle. Let us suppose that the rod F has an absolute length of exactly 3 m. at 0° C., and that the line of sight is 3 m. above the foot of the rod as in Figs. 1 and 2 above. At 30° C. the rod is 0.36 mm. longer, and the rod reading at that temperature is therefore 0.36 mm. too short. The temperature correction is then 0.36 mm., and the sign of the correction will be the same as the sign of the measured difference of elevation, unless the temperature is below 0° C.

Where the leveling is done over a section which is practically level the total correction for temperature of the rod is insignificant or even zero. Where the section changes much in elevation, however, the effect is a vertical base line measured in very short sections, and the rod lengths and coefficients of expansion must be known just as the lengths and coefficients of the tape lines on horizontal base measurements are known.

The temperature correction is computed from the following formula:

$$\text{Correction in millimeters} = \Delta E \times T \times 0.004.$$

Where ΔE is the difference of elevation in meters and T is the temperature of the rod at the time of observation in degrees C.

In this work the temperature correction for a single section was sometimes as much as 5.8 mm., or 0.019 ft.

The Index Correction—In the index correction, we take account of the fact that the zero of graduation and the foot of the rod are not exactly coincident. At times the index error is so small as to be practically zero, and is not applied except in the case where a reading is taken on a benchmark without the use of the rod. When the rods are in the field for a year or more the index error becomes appreciable on account of the wear on the bottom, and the correction must be entered for each section on the computation form.

The effect of the correction varies with the different ways in which the rods are used. For example, if two rods A and B, having index errors of a and b, respectively (a being greater than b) were used and the level section started from A as a backsight and ended on B as a foresight, the correction for this section would be (a—b).

As a and b are always negative they would have the effect, in this case, of decreasing the difference of elevation if the readings on rod A were the greater and increasing it if the readings on rod B were the greater. If the level section started from B as a backsight and ended on A as a foresight, the correction would have the opposite effect. This can be seen from a study of Fig. 2 (small Fig. 4).

If the index correction for both rods is the same there is no correction for the section. Also, if the number of instrument stations be even and all readings are taken on the rods, there is no correction for the section. If the number of instrument stations be odd, only the last station need be considered and the amount of the index correction is the difference of the index errors of the two rods. If only one rod be used as in ordinary leveling, there is no index correction.

The average wear per year on the bottom of the leveling rods was 0.3 mm. This wear was detected by measuring the first decimeter of each rod by means of a finely graduated caliper square. As the first decimeter graduation is marked by the fine lines on the silver plug inserted in the face of the rod, this measurement could be made very accurately and was repeated every two weeks.

COMPUTATION OF PRECISE LEVELS

Table III shows the form on which the computations are made. The computation is discontinuous, showing results from different sections to illustrate the method of applying the corrections. The forward line from B. M. 3 to B. M. 4 on this form is that for which the record is given in Table II.

The fourth column on the left (Table III) contains the sum of the rod intervals or stadia distances between benchmarks. The fifth column contains the distances between the benchmarks in kilometers and is derived from the fourth column by knowing that an interval of 300 mm. subtended on the rod corresponds to 100 m. along the line, regardless of the lengths of the separate sights. The sixth column contains the difference between the backsight and foresight intervals for each line, and in the case of the first line shows the foresight distance to be 2 m. longer than the backsight distance.

The corrections for curvature and refraction shown in the second columns on the right-hand side are those due to the slight differences of corresponding foresights and backsights, no correction being necessary when the corresponding sights are exactly equal.

In applying the curvature and refraction correction to the level results, only the difference of rod intervals of the individual setups is considered, not the difference of rod intervals of the entire section. The correction is usually inappreciable and seldom exceeds 0.1 mm. under actual conditions.

It is important to note that this is in the main a correction for curvature, a quantity which is not uncertain, the uncertain refraction correction being on an average about one-eighth as great as that for curvature. Curvature and refraction errors exist in every line of sight, but are eliminated if the foresight and backsight distances are kept equal.

The level correction shown in the third column on the right is equal to the constant C (previously defined) times the value in the sixth column of the left. Its sign is fixed by the signs of the two factors. In the forward line from B. M. G. to B. M. W. the value of C was +0.007 and gave a correction for the level of 0.2 mm. This correction will very seldom exceed 0.3 mm. under actual conditions.

The fourth column on the right gives the correction due to the excess of length of the rods at 0° C. For the first five completed lines the rods A and C were used, their mean length being 0.133 mm. too long on each meter. It will be seen from an inspection of the corrections in this column that the amount of the correction will vary with the measured difference of elevation, provided the correction per meter is the same for all rods.

In the line from B. M. 41 to B. M. 42 the difference of elevation was over 34 m., and as the mean length of the rods at 0° C. was 0.133 millimeter too long on each meter, the length correction for that section is 4.6 mm. or 0.015 ft. This is no inconsiderable correction in precise leveling. Again, in the line from B. M. G. to B. M. W., the difference of elevation was over 57 m. and there is no length correction; that is because the mean length of the rods used on that particular section is exactly 3 m. at 0° C.

The next column gives the correction due to the expansion of the rods from 0° C. to the temperature of observation, computed with the known coefficient of expansion of the rods, namely, 0.000004 per degree centigrade. Here also we see that the amount of the correction varies with the measured difference of elevation, the greatest correction be-

TABLE III. FORM OF COMPUTATION OF PRECISE LEVELS

Line: So. Ferry to Harlem Ship Canal Manhattan Borough New York City, N. Y. Observer: F.W.Koop Year—1911

B. M's	F. or B.	No. of stations	Sum of rod intervals mm.	Distance in km.	Rod intervals 2B — ΣF mm.	Mean rod readings ΣB m.	Mean rod readings ΣF m.	Approx. diff. of elev. m.	Mean temp. of rods, °C	Corrections: Index corr. mm.	Corrections: C. and R. mm.	Corrections: Level mm.	Corrections: Length of rod mm.	Corrections: Temp. of rod mm.	Difference of elevation: Each line m.	Difference of elevation: Mean m.	B — F Divergence mm.	Toward or from sun	S. or C.	Wind	Date and hour
1-2....	F.	12	2518	0.856	— 6	18.4590	15.4062	+ 3.0528	17	0.0	0.0	0.0	+ 0.4	+ 0.2	+ 3.0534	+ 3.0535	+ 0.3	L.	S.	ML.	9/28–1 : 30
	B.	13	2618		— 14	17.2293	20.2824	— 3.0531	15	0.0	0.0	0.0	— 0.4	— 0.2	— 3.0537				C.	C.	2 : 45
2-3....	F.	14	2098	0.714	— 5	23.8577	17.7506	+ 6.1071	11	0.0	0.0	0.0	+ 0.8	+ 0.3	+ 6.1082	+ 6.1080	— 0.3		C.	C.	12/10–8 : 30
	B.	14	2186		+ 4	17.7193	23.8261	— 6.1068	13	0.0	0.0	0.0	— 0.8	— 0.3	— 6.1079				C.	C.	12 : 00
3-4....	F.	7	1440	0.478	+ 14	11.8189	11.1353	+ 0.6836	9	0.0	0.0	0.0	+ 0.1	0.0	+ 0.6837	+ 0.6830	— 1.3		C.	C.	12/10–9 : 30
	B.	7	1427		+ 7	11.1766	11.8580	— 0.0823	14	0.0	0.0	0.0	— 0.1	0.0	— 0.6824				C.	C.	11 : 30
G-W....	F.	57	9649	3.179	+ 31	63.9932	121.8646	— 57.8714	21	0.0	0.0+0.2		0.0	— 4.6	— 57.8758	— 57.8761	— 0.6	L.	S.&C.	C.	6/13–10 : 45
	B.	55	9427		+ 15	117.0606	59.1900	+ 57.8706	25	0.0	—0.1+0.1		0.0	+ 5.8	+ 57.8764			Tl.	S.&C.	MR.	6/14–11 : 30

TABLE IV. FORM SHOWING ABSTRACT OF SPIRIT-LEVEL RESULTS

Borough: Manhattan Instrument: Level No. 1. Rods: A. & C. Observer: F. W. Koop Computers: F. W. & J. S. Lowery

Date	From B.M. to B.M.	Distance in km.	Difference of elevation: Forward line	Backward line	Mean	Discrepancy: Partial	Total accumulated	No. of B.M.	Distance from B.M. No. 8	Adjusted difference of elevation	Elevation above mean sea level		
1911		m.	m.	m.	m.	mm.	mm.	U.S.	km.	m.	m.	ft.	
							−0.4	H	14.278		2.4382	7.999	On seawall at Governor's Island
Sept. 25–26	H-1	0.947	−0.2923	+0.2923	−0.2923	0.0	−0.4	1	15.225	−0.2922	2.1460	7.041	
Sept. 28	1–2	0.856	+3.0534	−3.0537	+3.0535	+0.3	−0.1	2	16.081	+3.0536	5.1996	17.059	
Dec. 10	2–3	0.714	+6.1082	−6.1079	+6.1080	−0.3	−0.4	3	16.795	+6.1080	11.3076	37.098	
Dec. 10	3–4	0.478	+0.6837	−0.6824	+0.6830	−1.3	−1.7	4	17.273	+0.6830	11.9906	39.339	
Dec. 10	4–5	0.136	+1.6464	−1.6461	+1.6463	−0.3	−2.0	5	17.409	+1.6463	13.6369	44.740	Center of E. cut, S.W. cor. City Hall

ing +5.8 mm. or 0.019 ft. for the backward run of the line from B. M. G. to B. M. W.

The sum of the quantities in the fourth and fifth columns in any line gives the correction due to the excess of length of the rods at the temperature of observation.

The sixth column gives the result for each line after the corrections have been applied, and the seventh column the mean difference of elevation for each completed line. The next column gives the divergence between the backward and forward lines for each section.

The last four columns on this form are for use whenever special studies are to be made to determine if possible the sources of the principal errors of leveling. It should be noted that the times of the backward and forward runnings of any section, as indicated in the last column, have no fixed relation to each other. The two runnings are sometimes made on the same day, sometimes on different days.

Table IV shows the abstract of results which is the form actually used in collecting the results of the computation form shown in Table III. While the computation is discontinuous, showing results from different sections, the abstract is continuous, and is the form in which the final results appear.

The most interesting columns on the left are those showing the discrepancies. In the seventh column the discrepancy between the backward and forward running of each section is shown, and from these quantities the probable error for any section of the level line may be computed. In the next column the total accumulated discrepancy from the beginning of the line, which in this case is the Coast Survey tide staff at Fort Hamilton, is shown.

At the head of the column it is seen that the sum of the discrepancies from Fort Hamilton to Governors Island is —0.4 mm. for a distance of more than 14 km., or at the rate of —0.029 mm. per km. At the bottom of the form the computed rate of accumulation of discrepancy between the forward and backward lines was —1.9 mm. on 30 km. or only —0.06 mm. per km. This is a remarkably small error and proves conclusively that there is no systematic error in the leveling.

The third column on the right gives the adjusted difference of elevation for any section, from which the elevations are at once derived. This column is not shown on the Coast Survey form of abstract, but it fits in very well and keeps all the figures before the computer. An examination of the quanities showed that the largest correction applied in order to close the several circuits of which this line forms a part, was at the rate of 0.038 mm. per km. or at the rate of 0.0002 ft. per mile. This correction being so small, it was at once disposed of by adding 0.1 mm. to a few of the longer lines.

The fourth and fifth columns contain the resulting elevations in meters and in feet respectively. The last column is for brief descriptions of the benchmarks, a few of which are shown.

RIVER CROSSINGS

All river crossings exceeding 500 ft. in width were made by the simultaneous reciprocal method with uniformly good results. There were ten river crossings all told, eight of them ranging in width from 790 ft. at Elizabethport, on the Kill Van Kull, to 4400 ft. at the Narrows. The two remaining crossings were those at Newton Creek and at the Harlem River; these streams being less than 500 feet in width at the points crossed, were leveled over directly with one instrument, repeating the operation several times at each side. In such cases the individual results for difference of elevation between the benchmarks located on opposite shores were well within the limit of error.

Before any river crossing was made each level was tested and put in very nearly perfect adjustment by means of the method previously described for the determination of C.

Fieldwork—Two levels and two rods were used, one of the levels being set up on each side of the river about 20 ft. distant from the B. M. or T. P. A rod reading was then taken by each observer on the B. M. or T. P. nearest his instrument; then, at a given signal the observations were commenced and the targets were set on each of the distant rods at nearly the same instant, and the readings recorded. Simultaneous observations were therefore made in opposite directions by each observer.

This operation was repeated as a rule ten times, allowing about five minutes between each reading so as to permit of a change in the atmospheric conditions. The near rod was read only at the beginning and at the end of the program of observation. The observers then exchanged stations, each man taking his instrument with him; the near rods were again read, and at a prearranged signal the observations on the distant rods were repeated in the opposite direction exactly as before. The arithmetical mean of each set of readings was taken as the final result.

When employing this method, it is very essential that both the instruments used should be very nearly alike in the magnifying power of their telescopes and in the sensitiveness of their spirit levels. On work of this kind the very sensitive bubble in the level vial of the geodetic level reveals the true qualities of the instrument, and exceptionally close readings can be made. It is also essential that the two targets be set as nearly simultaneously as possible in order that the same atmospheric conditions may affect both sights.

Record Forms—Table V and Fig. 3 illustrate a simultaneous reciprocal level crossing of the East River from Astoria to East River Park, and show better than can be described the methods employed and the reasons therefore. The lines of sight as shown in the illustration are, of course, exaggerated somewhat.

The heavy curved line is a line of true level, being parallel to the surface of still water and therefore at all points equidistant from the center of the earth. If the lines of sight through each instrument were lines of true level, they would take the position of the broken lines in the illustration; not being lines of true level they are shown by the use of full lines as they really are, normal to the radius of the earth through the center of each instrument. In this case, we desire to obtain the difference of elevation between B. M. 149B and B. M. 149C, the distance between the B. M.'s. being 566 m. or nearly 1927 ft.

When making the river crossing, it is best to consider the observations made from both sides of the river as being made in the forward direction as indicated by the arrow; that is, to consider the elevation in both cases as being carried from B. M. 149B to B. M. 149C so as to give two forward runs, instead of one forward and one backward run as in ordinary leveling. This simplifies the computation, gives a better understanding of the work and causes less confusion in the signs.

On this crossing level No. 1 is set up near B. M. 149C, and level No. 2 is set up near B. M. 149B as shown in the illustration. A backsight is taken on B. M. 149 B with level No. 2, and then a foresight is taken on B. M. 149C with the same instrument.

The resulting difference of elevation between the two B. M.'s being uncorrected for curvature and refraction will be in error by the amount ab, which is the amount of the correction necessary to bring the line of sight down to the line of true level. In other words, the elevation of B. M. 149C as derived from B. M. 149B with level No. 2 will be too low by the amount ab. For the same reason, the difference of elevation between B. M. 149B and B. M. 149C as derived from observations with level No. 1 on the west side of the river, will be in error by the amount a'b', which is the sum of the curvature and refraction corrections for the line of sight through level No. 1, and the elevation of B. M. 149C will be too high by the amount a'b'.

As the observations taken with both instruments were simultaneous, the same atmospheric conditions affect both lines of sight, and ab = a'b', and the actual difference of elevation is the mean of the two determinations. The operation is therefore a very simple one, and the reduction is similar to the application of a constant quantity to both backsight and foresight, which does not affect the true difference of elevation.

After a set of ten readings has been taken with each instrument at each station, the mean of the last two deter-

TABLE V. COMPUTATION OF RECIPROCAL LEVELS ON RIVER CROSSINGS

Line: Astoria to East River Park — New York City, N. Y. — Observers: F.W.K.—J. S. — Year—1911

B. M's.	Position of instrument	F. or B.	Distance in kilometers	Mean rod readings m.	Mean rod readings m.	Approx. diff. of elev. m.	Mean temp. of rods °C.	Corrections Index corr. mm.	Corrections Length of rod mm.	Corrections Temp. of rod mm.	Difference of elevation Each line m.	Difference of elevation Mean m.	S. or C.	Wind	Date and hour
149b–149c	Level No. 2, on E/S river....	F.	0.566	1.8450	0.9904	+0.8546	5	0.0	+0.1	0.0	+0.8547	+0.8827	C.	C.	12/20–10 : 00
149b–149c	Level No. 2, on W/S river...	F.		2.4817	1.5710	+0.9107	5	0.0	+0.1	0.0	+0.9108		C.	C.	2 : 00
149b–149c	Level No. 1, on W/S river...	F.	0.566	2.6169	1.6910	+0.9259	5	0.0	+0.1	0.0	+0.9260	+0.8820	C.	C.	12/20–10 : 00
149b–149c	Level No. 1, on E/S river....	F.		1.9755	1.1375	+0.8380	5	0.0	+0.1	0.0	+0.8381		C.		2 : 00
								Mean for both levels =				+0.8824			
149b–149c	Level No. 2, on W/S river...	F.	0.566	2.4752	1.5720	+0.9032	8	0.0	+0.1	0.0	+0.9033	+0.8867	C.	C.	12/21–1 : 00
149b–149c	Level No. 2, on E/S river....	F.		1.9048	1.0348	+0.8700	8	0.0	+0.1	0.0	+0.8701				3 : 00
149b–149c	Level No. 1, on E/S river....	F.	0.566	2.0304	1.1624	+0.8680	8	0.0	+0.1	0.0	+0.8681	+0.8860	C.	C.	12/21–1 : 00
149b–149c	Level No. 1, on W/S river...	F.		2.6121	1.7083	+0.9038	8	0.0	+0.1	0.0	+0.9039		C.	C.	3 : 00
								Mean for both levels =				+0.8863			
								Final difference of elev.				+0.8843			

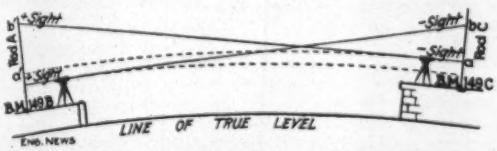

FIG. 3. DIAGRAM OF RECIPROCAL LEVELING ACROSS RIVER

minations of the difference of elevation is combined with the mean of the first two determinations, and the final mean is the accepted difference of elevation between the two benchmarks.

Precision of Observations—In order to obtain a measure of the precision of the observations, each rod reading is subtracted from the mean of its set and the algebraic differences are recorded. These differences are the residuals from the mean and are called the residual errors. The algebraic sum of the residuals must always equal zero.

An inspection of the residuals will show how much each reading differs from the mean, and these differences must be limited. On river crossings from 600 to 800 m. in width, any reading differing more than 1 cm. from the mean would be rejected. From the sum of the squares of the residuals we obtain the probable error of a single observation and also the probable error of the mean.

When the difference of elevation between the two benchmarks on opposite sides of a river is very small, it is especially necessary to be careful of the signs; for, even though both crossings are assumed to be made in the forward direction, one of the values may be plus and the other minus. In such a case, the true difference of elevation is the algebraic sum of the two differences of elevation divided by two. No separate abstract of results is necessary for this work, these being incorporated in the usual abstract covering that territory.

Conclusions on the Accuracy of Reciprocal Leveling—The best time of day for reciprocal leveling is between 10 a.m. and 3 p.m., as between these hours the vertical refraction is less variable than at other times of the day. On a wide crossing, observations should be taken on at least two days so as to average up the atmospheric conditions. The higher the line of sight above the water, the more accurate the results are apt to be.

It has been frequently stated that a river crossing with sights 1 km. long or longer would be much weaker than the remainder of a precise level line, even if the best of targets were used and reciprocal simultaneous sights were taken. It can be here positively stated, however, that the results of river crossings made on different days by the method above described do not as a rule vary beyond the limit of error fixed for land leveling, and that wherever these crossings form part of level circuits large or small that portion of the circuit has proven to be as strong as the remainder. In fact, some of the best circuit closures are those representing both large and small circuits containing one or more river crossings.

A number of engineers have expressed the opinion that river crossings would be more accurate if the method of trigonometric leveling were employed. In no case can this work attain a degree of precision comparable with that done by spirit leveling. The Coast Survey has done a large amount of trigonometric leveling in connection with its triangulation work, and in order to prevent an accumulation of errors in the elevations thus determined, connection is made at various points with precise level benchmarks, and the trigonometric leveling is adjusted to fit the precise leveling between these points. Vertical angles for determining differences of elevation are usually very small and a large probable error may occur in a computed elevation, even when the probable error of the vertical angle is not large.

THE NEW YORK CITY LEVEL NET

Fig. 4 shows the Board of Estimate precise level net. This system extends from Perth Amboy, N. J., to Dobbs Ferry, N. Y., encircling and traversing each of the boroughs en route, as shown by the full heavy lines on the map (Fig. 4).

Datum Plane—In 1881 the U. S. Coast Survey transcontinental line of spirit levels was commenced at the Sandy Hook tide-gage and continued westward. The elevations of this line were based upon continuous tidal observations made with a self-registering tide gage during the six years 1876-

FIG. 4. MAP OF GREATER NEW YORK, SHOWING PRECISE LEVEL NET

1881 inclusive. The probable error of the mean sea level determination resulting from these tidal observations is +9.1 mm.

In 1886-87, the Coast Survey ran a precise level line from Sandy Hook, N. J., to Dobbs Ferry, N. Y., and the elevations along this line were based upon the same tidal observations at Sandy Hook as were used in connection with the transcontinental line. The Coast Survey precise level line of 1886-87, from Sandy Hook to Perth Amboy is shown as a dotted line on the map, and the few benchmarks that are left, from Perth Amboy to Dobbs Ferry, are shown as heavy stars.

When the running of the precise level lines in New York City was commenced by the Board of Estimate it was decided

to determine whether or not the old benchmarks established by the Coast Survey in 1886-87 had changed in elevation. It was also decided to ascertain whether mean sea level at Sandy Hook was at the same elevation as mean sea level at Fort Hamilton, as determined by the long series of tide observations taken there by the Coast Survey.

While it had been intended in the beginning to base the elevations resulting from the Board of Estimate leveling upon the elevations of the existing Coast Survey benchmarks, a number of discrepancies were developed as a result of the first season's leveling and it was then decided not to fix any elevations until all the old Coast Survey benchmarks had been connected and the results examined.

The Board of Estimate system of precise level lines begins at a benchmark in Perth Amboy, N. J., the elevation of which, above mean sea level at Sandy Hook, had been established in 1887 by the Coast Survey. This benchmark had been connected with the transcontinental line of spirit levels and is also a benchmark of the State Geological Survey.

Starting from this benchmark the level line crossed the Arthur Kill to Tottenville, and proceeding along the easterly side of Richmond Borough to Fort Wadsworth, crossed the Narrows to Fort Hamilton and connected with the Coast Survey tide-staff. When the resulting difference of elevation was applied to the benchmark at Perth Amboy, a discrepancy of 4.0 mm. was developed. As the probable error of the determination of mean sea level at Sandy Hook is ± 9.1 mm., the discrepancy of 4.0 mm. developed at Fort Hamilton tide staff is well within the probable error, and it can reasonably be assumed that mean sea level is the same at both places.

Having reached this conclusion, the Board of Estimate level lines in the Borough of Richmond were adjusted to a datum between the initial benchmark at Perth Amboy and mean sea level at Fort Hamilton, as determined from 19 years of continuous tide observations by the U. S. Coast Survey.

The Precise Level Net—The large circuit in the Borough of Richmond embraces almost the entire borough and is divided into five smaller circuits, one of the dividing lines passing through the highest land in the city. The largest circuit has a perimeter of 67 km. or 42 miles and failed to close by 6 mm. or 0.02 ft. The peculiar loop in the westerly part of the borough may need some explanation.

The reason for the loop is that the Fresh Kills lie just west of it and although Bridge Avenue runs across the Kills, it is a narrow road with a tar like pavement adorned only by telegraph poles and affords no sites whatever for permanent benchmarks. As a good route for the level line ran through New Springville and Richmond and the country was liberally supplied with sites for good benchmarks, the connection between Linoleumville and Fresh Kills was made by way of these villages.

It will be seen from the map that the level lines extend easterly to Little Neck, Floral Park, Springfield, Cedarhurst, Far Rockaway, and Belle Harbor at Rockaway Beach. At Seaside, the level line connects with the Department of Docks and Ferries tide-gage in Jamaica Bay. On the north the level lines embrace the towns of Yonkers and Mount Vernon, passing through such country as afforded the best sites for permanent benchmarks.

The small heavy triangles shown on the map indicate the Coast Survey triangulation stations for which elevations were established by the Board of Estimate leveling. The peculiar convention just west of Central Park indicates the position of a Coast Survey astronomic station in the grounds of the American Museum of Natural History. Its elevation was also determined.

Benchmarks consist either of marks on buildings or ground monuments. The marks on buildings are generally of the set-bolt type. The monuments are stone or concrete with copper bolts.*

Changed Elevations of Old Benchmarks—Many important and interesting facts were developed as a result of the Board of Estimate leveling. The most noteworthy of these is the fact that many of the Coast Survey benchmarks established in 1886-87 have apparently changed materially in elevation. These changes have occurred no doubt on account of the settlement of the buildings on which the benchmarks were placed, and not, in the opinion of the author, on account of coastal subsidence.

Instead of holding these historic benchmarks and thus vitiating the high precision of the Board of Estimate leveling, new elevations were established for most of them. The amount that these benchmarks have settled and a study of these differences prove how unwise it would have been to hold the old elevations established by the Coast Survey.

*The type of benchmarks used in this work was described by Mr. Koop in "Engineering News," Sept. 21, 1911, p. 344.

Another important fact developed is that the old T cut benchmark at the southwest corner of the New York City Hall is almost 3 in. lower than its old elevation indicates. When this benchmark was established in 1840 its elevation was fixed at 24.413 ft. above the old monument in the grounds of Bellevue Hospital.

It may be argued that the monument has settled; this is disproved by the fact that the monument still maintains its exact relation to the original line of benchmarks established along 26th St. in 1880. The monument could not therefore have settled unless all of 26th St. has settled also and by the same amount.

When this difference in the elevation of the old City Hall benchmark was made known in the early part of 1912, many of the older city engineers were severely shocked. The existence of this discrepancy, however, has since been proved by a number of the city departments.

Several of the existing datum planes used by various city departments are affected, and it is more than likely that a new datum plane chart based upon actual surveys will be published in the report on the Board of Estimate leveling.

Amount of Fieldwork—On this work 340 miles of double or completed level lines have been run, each line being leveled over at least twice in opposite directions. The percentage of level line run more than twice was 3. There were 1500 benchmarks touched on for which elevations will be established, or about four to each mile of completed level line.

COMPUTATIONS

The level nets were adjusted by the method of least squares. As the leveling was of the same precision throughout, the computations were very much simplified. The largest level net contained three closed figures involving five river crossings and had a perimeter of 118 km., or 74 miles. This large circuit passes through four of the boroughs and the error of closure amounts to 3.8 mm., or 0.012 ft.

A circuit from Long Island City to Little Neck, L. I. and return via Flushing, Whitestone, Bayside, Little Neck, Floral Park, Hollis, Jamaica and Glendale, failed to close by 6 mm., or 0.02 ft., the length of the circuit being 64 km., or 40 miles. Another circuit of 83 km., or 52 miles, which embraces almost all of Queens Borough, failed to close by 1 cm., or 0.033 ft. A small circuit of 17.6 km., or 11 miles, embracing Flushing, College Point, Whitestone, Willets Point and Bayside, has a closing error of 0.1 mm., or 0.0003 ft.

In the adjustment of the level nets some of the corrections applied were so small that it would not have made any material difference if any other method had been followed. The greatest correction so far applied to close any circuit, large or small, was at the rate of 0.6 mm. per km., and this was for a very small figure.

The probable error of the mean result for a section was computed in millimeters by the formula

$$r'' = 0.674\sqrt{\frac{\Sigma d^2}{4s}}$$

in which d is the discrepancy between the forward and backward leveling over a section and s is the number of sections. The probable error of the most direct level line from Perth Amboy to Dobbs Ferry computed from this formula is ± 0.30 mm. and the total distance is 94 km., or 58.7 miles.

A Heavy Industrial Locomotive has been built for use on the 36-in. gage industrial railway system at the steel works of the Tennessee Coal, Iron & Railroad Co., at Ensley, Ala. It is a tank engine of the 0:6:0 class, with large tanks alongside the boiler and with a coal bunker on one side of the cab so that the cab is open at the rear. The wheels are placed close together, with a wheelbase of only 9 ft., to enable the engine to operate on sharp curves. The connecting-rods drive the rear axle, and the Walschaerts valve-gear is used to operate the slide valves. A steam brake is applied to all the wheels. The locomotive was designed for heavy and severe service, and is believed by the builders to have the heaviest load per square foot of track for an engine of this class, with its short wheelbase. The engine was built by the H. K. Porter Co., of Pittsburgh, Penn., and its leading dimensions are as follows:

Type of engine	0:6:0 (tank)
Gage of track	3 ft. 0 in.
Service	Yard work at steel plant
Wheels (6) diameter	3 ft. 10 in.
Wheelbase	9 ft. 0 in.
Weight in working order	147,000 lb.
Weight per sq.ft. of track	5.445 lb.
Cylinders	19x24 in.
Boiler pressure	200 lb.
Length over bumpers	27 ft. 8 in.
Width over all	8 ft. 10 in.
Height, rail to top of smokestack	13 ft. 0 in.
Water in tanks	2000 gal.
Coal in bunker	1200 lb.
Tractive force	32,000 lb.

Cleaning Stone Ballast by Screening

By W. I. Trench†

Experiments with screens instead of forks for cleaning stone ballast were made on the Baltimore & Ohio R.R. in 1912, using a form of screen developed by the writer and A. G. Zepp. The present method involves one of the most expensive and tedious operations in railway maintenance, and for this reason the cleaning is often deferred much longer than good practice would demand. It was felt that a great saving could be made by using a screen which would make a proper separation of stone and dirt and at the same time dispose of these two materials in a way to avoid further handling; with a single cast of the shovel instead of the repeated sifting motion and the further shoveling of the dirt in its disposal as required by the fork.

It was found that on double track the most efficient work was done by a gang of 12 men with three screens, one on each berm and one in the center ditch. The screens are alike, and rest on separate legs. The legs for use on the

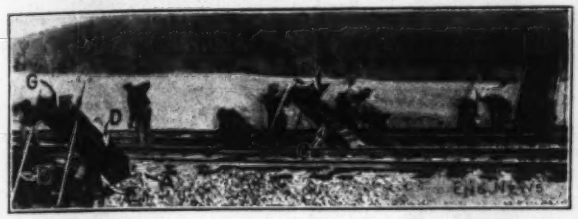

Fig. 1. Screening Stone Ballast on the Baltimore & Ohio R.R.

[(A) Legs supporting screen on berm; (B) adjustable support for top of screen; (C) screen in center ditch; (D) solid plate at back of screen; (E) chute delivering dirt to wheelbarrow; (F) top hood, as used on center screen; (G) top hood as used on side screens.]

berm are so designed that the screen rides on the ends of the ties, clear of trains, and deposits the cleaned stone in final position. The dirt is deposited into a wheelbarrow standing on subgrade. This is all shown at (A), Fig. 1.

The legs for use in the center ditch ride on the cleaned subgrade, and are of such a height that the dirt is deposited in a handbarrow placed beneath the screen, and the clean stone is left in the center ditch. The upper end of the screen is carried on supports (B), adjustable in height. The screen in the center ditch (C) is laid flat on the passage of trains, and lies wholly below the top rail.

The screen is set in a frame of 2x3 in. angles, with the larger leg upstanding. The screen is formed of ¼-in. rods, crimped together, giving a mesh ¾x8 in. With this mesh and with the screen inclined at 45° the separation of stone and dirt was perfect even in damp weather, and this cannot always be said of the results secured from forks. These crimped rods are set in a rectangular frame of 1x½-in. channels, bolted inside the main frame so that the screen can be detached for repairs.

The screen is backed with a galvanized iron slide (D), which gathers the dirt coming through and deposits it in a receptacle by means of a spout (E). The spout is really a hinged door suspended at its outer end by a chain and fastened. With this door closed the screen will hold about one wheelbarrow load of dirt, so that the operation is not stopped while the dirt is being dumped.

At the top of the screen is a hood (F) which is used in the position shown when in the center ditch, forming a deflector for the ballast thrown over the top. The method of operation in this case is to slide the screen backwards from the cleaned ballast towards the uncleaned ballast, the latter being thrown over the top and being left in clean condition at the bottom. When this screen is in use on the berm, the hood is thrown back, as at (G), and forms the top against which the ballast is thrown.

With each screen is a galvanized handbarrow that fits upon the horizontal legs when in use on the berm. It is so placed after a sufficient quantity of cleaned ballast has been allowed to fall outside the rail, the remainder being caught in the handbarrow and drawn across the rail to be deposited in the cribs. When in use in the center ditch, the handbarrow is placed beneath to catch the dirt.

The cross-section of the finished work is shown in Fig. 2. The cribs are cleaned to the bottom of the ties, the center ditch 18 in. below the top of rail, and the berm 24 in. below the top of rail at the end of tie and sloping to 3 ft. below top of rail at back of side ditch. Every 50 ft. one crib is cleaned to the bottom of the center ditch on one end and to the top of subgrade on the other, forming an outlet for water.

The dirt from the ballast is dressed upon the subgrade outside the ballast line, and in addition to giving a neat appearance serves to keep down weeds very effectually. What is not required for this purpose is used to widen embankments.

For the operation of three screens under ordinary circumstances, 12 men are sufficient; with long hauls of dirt, more men are added for wheelbarrow work. Of the 12 men, 2 shovel from each side of the berm onto their respective screens, 2 shovel from the center ditch onto the center screen, and 1 man in center of each track shovels from the cribs onto the screen most available. One man with pick advances ahead of shovelers to loosen hardened ballast before their arrival. These are dispensed with if ballast is loosened by means of a plow attached to work engine. Long stretches of ballast can be loosened in this way in a short time by work engine; enough to keep a gang going several days.

The remaining three men are sufficient usually to handle wheelbarrows in disposal of dirt, dress dirt down on berm and fork a uniform ballast line, although for a hand-laid ballast line more men would be necessary. By careful handling of this gang, ballast and dirt are disposed of at one operation in their final position and no further attention is necessary. In most cases it is found that the cleaning of ballast so reduces its volume that additional stone is nesessary. In this case the disposition of the stone from the screens is so handled that the berms and center ditch are filled out completely, and any deficiency occurs between the rails where additional stone can be most conveniently distributed from ballast cars without further handling. The gang of 12 men cost per day: Foreman, $2.40; 11 laborers at $1.60, $17.60; total $20.

A gang equipped and organized as above will cover 165 ft. of double track per day of 10 hrs, making the cost per mile of double track, $640. This includes cleaning ballast,

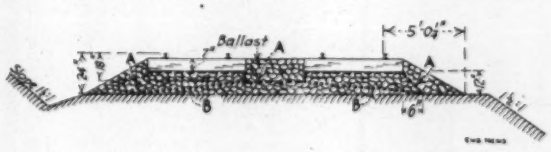

Fig. 2. Roadbed Cross-Section Showing Arrangement in Screening Ballast

(All ballast outside of upper dotted line (A) is removed, screened and replaced. At intervals of 50 ft. the ballast between two adjacent ties is removed to the lower dotted line (B), to give drainage from the center ditch.)

dressing ballast and disposal of dirt complete. Single-track work would cost considerably less than half this amount, as there would be no center ditch. An average of 227 wheelbarrow loads of dirt were removed per 100 ft. of double track cleaned.

For comparison with the fork method, the identical gang used above was tried with forks, and advanced but 72 ft. per day. This also included the dressing complete disposal of dirt, it being necessary to shovel the latter in wheel and handbarrows. This shows a cost of $1466 per mile of double track.

Various figures are given from time to time on the cost of cleaning ballast. Some of them are very much less than the above, and we can only believe that this is occasioned by omitting the disposal of dirt and dressing road complete, or on account of cleaning to a less depth of track than indicated above, or perhaps a less thorough separation of stone and dirt. In many cases, a raise is given track and ballast put under without cleaning. In the above test, no raise was made. For the quality of work done, we do not believe the figures can be much lessened. This shows a saving of 56% by use of screens over forks.

The screen weighs about 325 lbs., and can be easily propelled along track by the two shovelers at work at the respective screens, and they should last for years. The trial lot, with handbarrows complete, cost $45 each. If made in quantity and ordered on competitive bids, the cost would be greatly reduced.

*Abstract of paper in the 1913 "Proceedings" of the American Railway Engineering Association, Chicago.

†Division Engineer of Maintenance-of-Way, Baltimore & Ohio R.R., Baltimore, Md.

The New Gladstone Wet and Dry Dock, Liverpool, Eng.

The port of Liverpool, England, the largest export and next to London, the largest import port in the empire, has been under development for many centuries but particularly has undergone a radical and continuous improvement since 1858 when the Mersey Docks & Harbor Board came into existence and assumed complete control of the port administration. It has so developed the harbor that at the present time from the new Gladstone dock at the north end of the system to the Herculaneum dock at the southern extremity there are about 6½ miles of docks and basins with a water area of 430 acres and 26 miles of dockage; while across the river at Birkenhead the docks have a total water area of 127 acres with some 10 miles of quays.

As is well known the tidal range in the Mersey River at Liverpool is nearly 28 ft., which necessitates locks at the entrance to all the docks where boats of any size are to be loaded or unloaded. Advantage has been taken of the necessity for locks at the entrance to the docks by making a number of the docks dry-docks which can be pumped dry for the inspection and repair of the hulls of vessels as well as wet docks where loading and unloading can take place.

In the port area there are 17 of these so called gra-

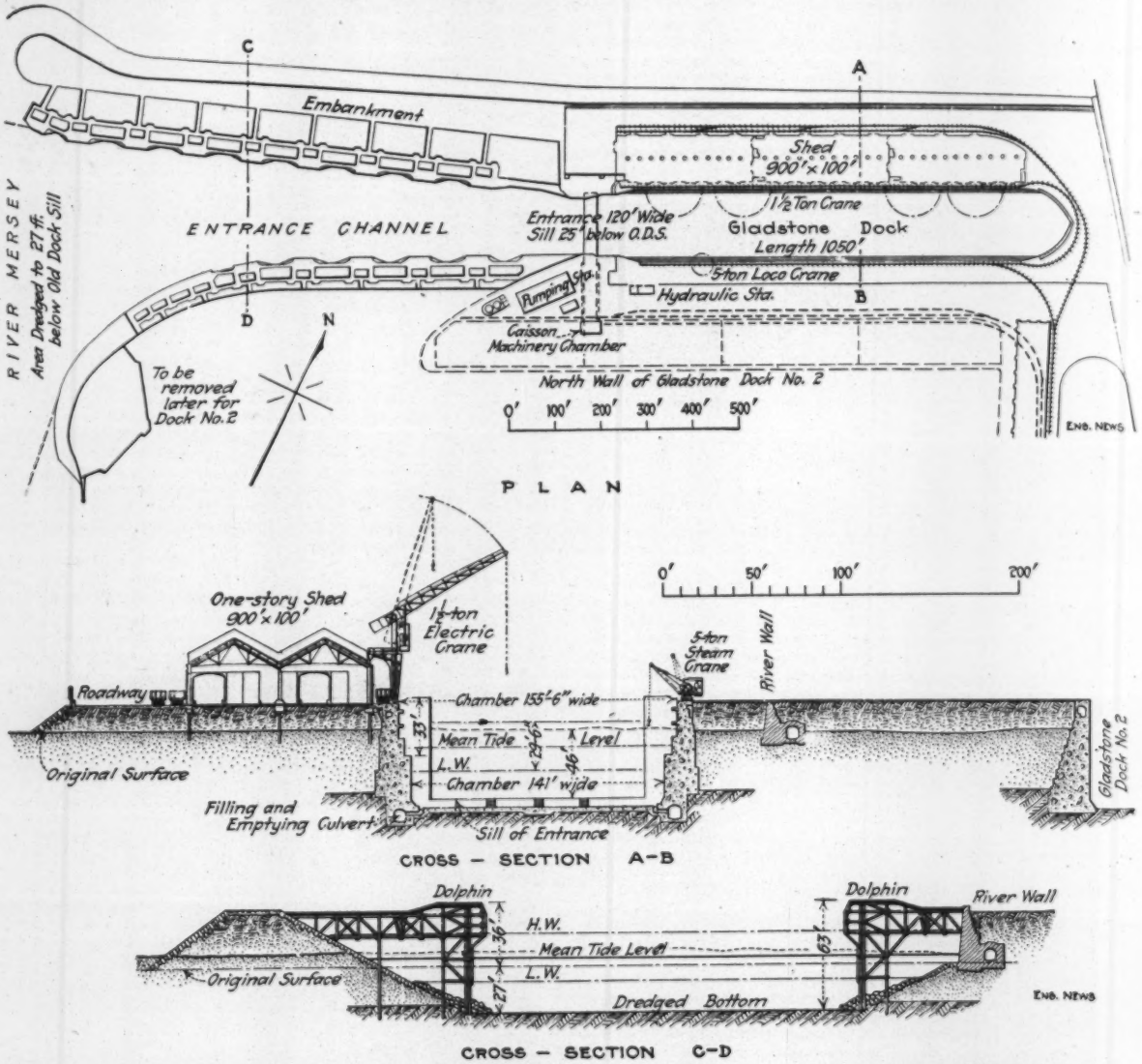

FIG. 1. PLAN AND SECTIONS OF THE GLADSTONE DOCK, LIVERPOOL, ENG., AT PRESENT THE LARGEST DRY DOCK IN THE WORLD

ing docks, of which the largest up to the present, the Canada dock, is 925 ft. long. There has just been completed within the past month the largest of these graving docks, known as the Gladstone dock and forming one of a series of docks built by the Board at the extreme northern end of the port. This Gladstone dock is the largest dry-dock which has ever been built, totaling 1050 ft. in length, 120 ft. width at the entrance, and an entrance sill depth which will allow 46 ft. over the sill at high tide and 35 ft. at the lowest tide.

The Gladstone dock development in its entirety was

designed by Anthony G. Lyster, then the chief engineer and now the consulting engineer of the Board, and was approved by Parliament in 1906. It was estimated then to cost about $15,000,000. On account of a depression in the shipbuilding trade at that time, none of the work was started until 1910, when the construction of the largest dock was commenced. This work has been going on continuously since that time and was completed in July of this year. The new development comprises two smaller half-tide docks which are entered by a lock, and the main Gladstone dock which has just been completed. In addition, lock entrances are to be built to the adjacent Hornby dock, which will provide four large berthing slips capable of taking the largest transatlantic vessels, and one dry-dock capable of taking the largest vessels now in existence or under projection.

Fig. 1 gives a plan and two sections of the recently completed Gladstone dock. As will be noted there it comprises an entrance channel (which is temporary and will be removed after the completion of the whole dock improvement) which is about 1200 ft. long and 400 ft. wide at its entrance, tapering down to the 120-ft. width at the dock proper. This latter has a chamber 155 ft. 6 in. wide at the top, and 140 ft. wide at the bottom, and provides a working chamber 120 ft. wide, 1020 ft. long and 46 ft. deep.

The entrance channel is formed by building up an embankment on one side and a retaining wall with a fill behind it on the other and dredging between them to give the requisite depth. Extending out from both embankment and river wall, as shown in one of the cross-sections in Fig. 1, have been built what are called in England "dolphins." These are timber guide walls marking the entrance channel and providing guides for the steamers entering the dock.

The section through the dock itself is shown also on Fig. 1. It will be noted that the lower portion of the interior wall is straight for a considerable depth, and starts to step back to form the altars for the graving dock only a few feet below the mean tide level. This section has been given in order to conform more nearly to the usual section of the larger steamers nowadays, so that they will not encounter any obstructions below water line when the dock is being used as a landing dock. The side walls are of solid concrete founded on bedrock, and the bottom of the dock is also of concrete 3 ft. 6 in. thick.

A sliding caisson closes the entrance, either excluding the water or maintaining its level inside the dock, as required. The use of gates was precluded by the consideration that they would have taken up an excessive amount of space. This caisson is of steel construction 124 ft. long on the outer face and 132 ft. long on the inner, with a width of 25 ft. The maximum head against it in still water will be about 46 ft. When in the closing position across the entrance its greenheart timber sills bear against the sill and jamb of polished granite, the pressure being taken by the riverside sill and jambs when water is impounded, and by the shoreside sill and jambs when the dock is being used for graving purposes. The caisson is moved by means of steel ropes actuated by electric motors placed on the masonry at the south end of the caisson recess, the outward pull being effected by a rope passing over pulleys on a cross-beam fixed in the south coping of the entrance. In addition to this caisson, a temporary floating or ship caisson is provided should it be necessary to remove the main caisson at any time.

The pumping machinery consists of five sets of centrifugal pumps with discharge pipes 54 in. in diameter, each of which is driven direct by a vertical four-cylinder Diesel engine, designed to run ordinarily at 180 r.p.m. and each capable of producing 1000 hp. The pumps are required to empty the dock of its whole contents, about 44,000,000 gal. on an 18-ft. tide in 2½ hr.

The well, situated on the south side of the dock near the entrance, forms a chamber 125 ft. long by 20 ft. wide and 25 ft. high. The water is led to it from a sump 22 ft. below the floor of the dock at its southwest corner, with two culverts between sump and well, each 8 ft. 6 in. by 12 ft. 6 in. in section. Some of the pumps are arranged to remove the drainage water from below the general level; two of them are also available for pumping water into the dock from the entrance to provide against any undue reduction of the depth of water owing to leakage or some other cause.

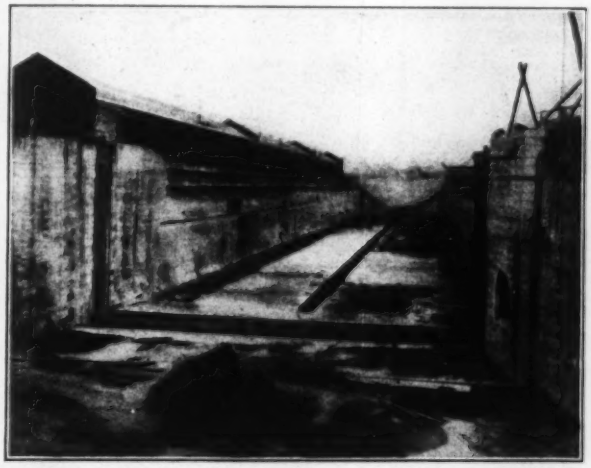

FIG. 2. VIEW FROM ENTRANCE OF GLADSTONE DOCK, APR. 16, 1913
(From London "Engineering," July 4, 1913.)

Alongside of the dock there is provided a one-story shed, 900 ft. long and 100 ft. wide. In front of it are four movable 1½-ton electric cranes and a 5-ton steam crane running on special rails along the sides and around the head of the dock. There are two hydraulic capstans at the ends, each capable of hauling 20 tons, and six 1-ton hydraulic capstans along the sides.

The dock is to be used for the large liners entering Liverpool harbor and is particularly intended to care for the new Cunard liner "Aquitania" which was recently launched. The entrance channels are to be dredged to give a depth of 48 ft. at high water in ordinary spring tide, so that as soon as the "Aquitania" is completed the dock will be ready for her accommodation.

The whole of the Gladstone dock work has been carried out by the Mersey Dock Board, under the direction, until the end of 1912, of Anthony G. Lyster, who is president-elect of the Institution of Civil Engineers and until 1913 was the Engineer-in-Chief of the Harbor Board. Since that date his successor, Thomas M. Newell, has been Engineer-in-Chief. W. Hemming Jones has been the resi-

dent engineer on the Gladstone dock works during its entire period of construction.

The dock was named not after the celebrated statesman, William E. Gladstone, but after Robert Gladstone, who, as chairman of the Mersey Docks & Harbor Board, did most valuable service.

Fig. 2 is a view taken last April showing the dock from its entrance toward the landside just before its completion.

Large Clam-Shell Dredges; Levee Building Methods and Standards in California

By Fred H. Tibbetts*

The editorial in Engineering News of June 5, entitled "Wanted—Machinery for Levee Building" and the subsequent correspondence show that in some respects the builders of the Mississippi River levees have been a little backward in adopting new methods of construction.

From the letter from Capt. C. O. Sherrill's office (June 26, p. 1340) and from other sources, it appears that consideration has not been given to the use of very large, though costly, clam-shell dredges upon which chief reliance is placed for levee construction in California.

In the Sacramento Valley this type of machinery has been developed until the largest machines can place material at one operation 300 ft. or more from the point at which it is excavated.

The clam-shell dredge "Hercules" of the Ajax Dredging Co., has a boom 205 ft. long with a clam-shell bucket containing 6 cu.yd. when level full. The hull is 70x140 ft. and the depth 12 ft. 10 in.

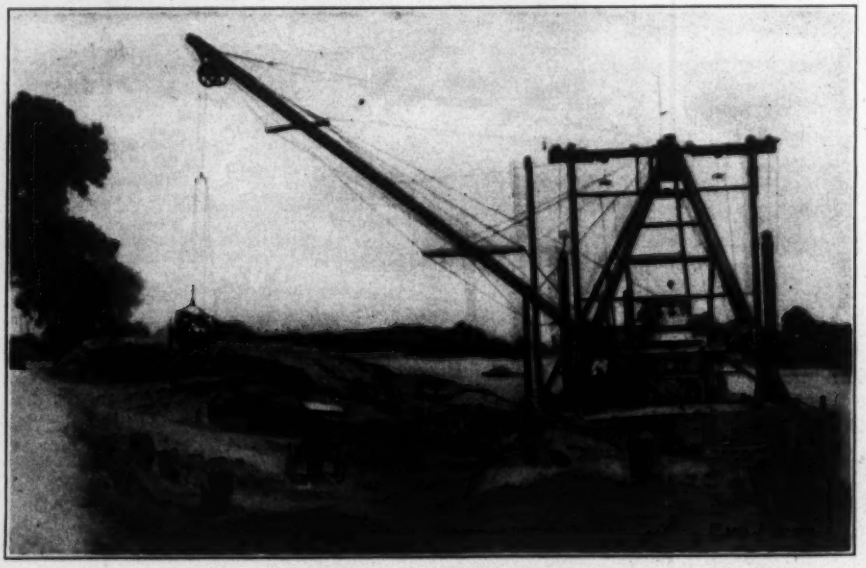

Fig. 2. Dredge "New Grand Island" Constructing Levee; 175-Ft. Boom, 5-Cu.Yd. Bucket

The new clam-shell dredge, "Netherlands," now under construction for the Netherlands Farms Co., which will be used in completing the back levees of Reclamation District 999 (a 30,000-acre tract in the lower Sacramento Valley), will be electrically operated and has a boom 220 ft. long, with a 6½-cu.yd. bucket, and, the writer believes, will be the largest of this type ever constructed.

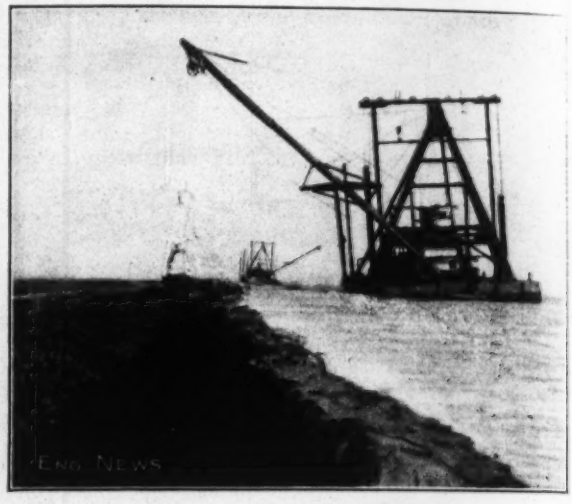

Fig. 1. Large Clam-Shell Dredge Reinforcing Sack Topping, Colusa Basin, Calif., at Crest of March, 1911, Flood

This machine will be able to deposit material about 307 ft. from the point at which it is excavated, and is designed to build a levee with side slopes of 1 on 3 and 1 on 4, with a top width of 6 ft. and a height of 26 ft. A 30-ft. berm can be left and the slope of the borrow pit limited to 1 on 2 and its depth to 8 ft.

These machines cost from $75,000 to $125,000, completely equipped, but they can build large levees at a cost, under favorable conditions, of 4 to 5c. per cu.yd.; under unfavorable conditions, at not over 8 to 10c. per cu.yd.; and they will handle from 75,000 to 150,000 cu.yd. per month. By 1915 there will probably be from $10,000,000 to $20,000,000 worth of levees constructed in the Sacramento Valley by means of such machines.

The writer, who has spent some time in studying the 1912 flood in the Mississippi River, is well aware that the use of this type of machinery might require some modification of the standard specifications now in force for the construction work on the Mississippi River levees.

It is unnecessary for such machines to reach farther than the rear of the levee crown, for dredges in any ordinary material can, if properly handled, construct the rear slopes from 1 on 3 to 1 on 6 as desired. During

*Of Haviland, Dozier & Tibbetts, Civil Engineers, Alaska Commercial Bldg., San Francisco, Calif.

high-water periods, the dredges could work while floating over the berm. At other times water for flotation could be provided by pumping.

It is quite usual in California to keep the water in the borrow pit, by means of pumping, at or near the

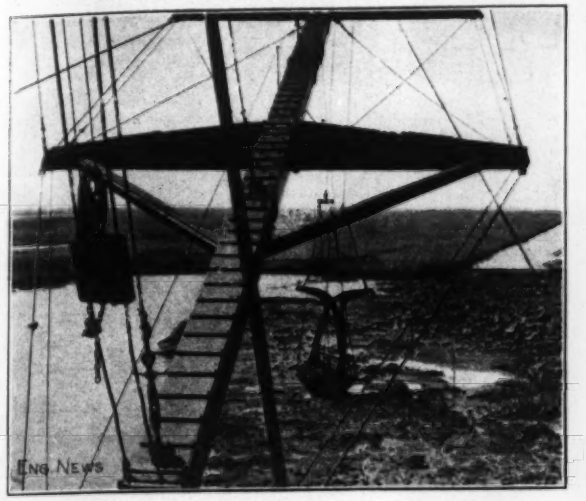

FIG. 3. BOOM CONSTRUCTION OF THE DREDGE "VULCAN"; 185 FT. LONG, 5¼-CU.YD. BUCKET WEIGHING 12 TONS

ground surface, in which case the depth of the borrow pit need not exceed 7 or 8 ft. for the largest machines. Frequently the water is kept above the ground surface by a small auxiliary levee constructed by a ditching machine, a trenching machine or by teams.

It is very awkward, although not necessarily impossible, to leave traverses of undisturbed material interrupting the continuity of the borrow pit. The writer's observation, however, of such traverses during floods seems to indicate that they are a dangerous source of weakness, rather than a protection to the levees.

As the material placed by floating clam-shell dredges is

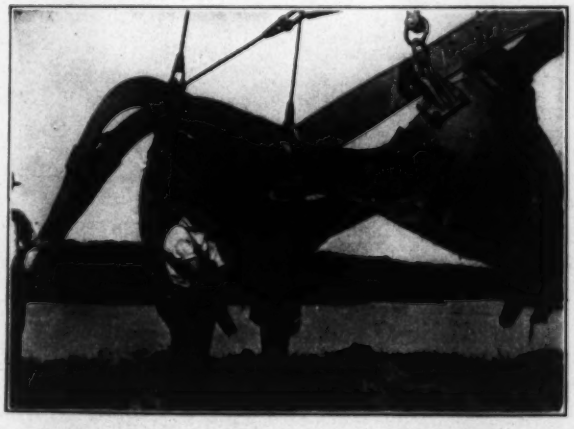

FIG. 4. A 12-TON 5¼-CU.YD. CLAM-SHELL BUCKET OF THE DREDGE "VULCAN"

usually saturated, and as it may be dropped on the levees from a considerable height (5 to 10 ft. or more) in masses of 2 to 6 cu.yd., the usual result is a very thoroughly compacted levee section (this last observation was suggested by A. M. Shaw's letter in your issue of June 26, p. 1339).

Suction dredges also are being used successfully within the last year for levee construction along the Sacramento River. Huge levees can be built at a comparatively small unit cost of sand pumped a quarter- to a half-mile. Levees of this sort have been found very efficient when protected from wave wash or direct-current scour.

The West Sacramento Co. is building a levee along the west bank of the Sacramento River in front of the terminal and industrial section 22 ft. in height with a river-

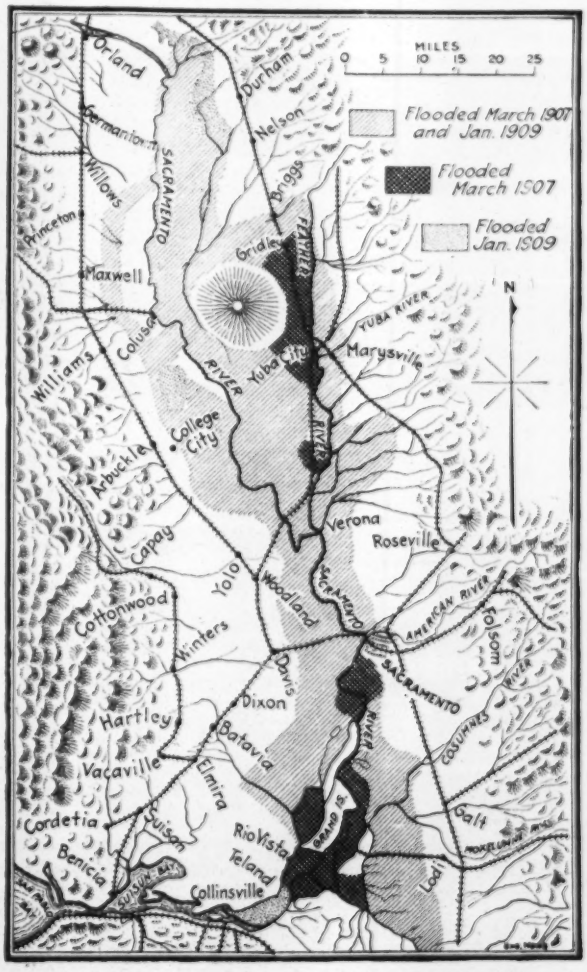

FIG. 5. MAP OF SACRAMENTO RIVER SHOWING FLOOD AREAS

(Reproduced from Bulletin 43, United States Weather Bureau.)

ward slope of 1 on 3, a crown width of 100 ft. and a rear slope of 5%. The 1 on 3 slopes are obtained by the use of "trap" pipe, and the 5% slope on the rear is obtained with great uniformity and ease without any special equipment.

The Natomas Co., on the opposite side of the river, is first constructing a deep trench along the axis of the levee with a large drag-line scraper, piling the excavated material in high, narrow ridges on either side and backfilling the trench and spaces between the ridges with sand dumped in from a suction dredge on the river. This

gives a sand core impervious to burrowing animals and a riverward slope of more resistant material.

The Mississippi River Commission has a splendid fleet of large suction dredges which annually handles a large amount of material in channel-improvement work. It would seem that much good might be accomplished if this material were deposited to reinforce the rear slopes of the levees instead of being wasted in adjacent parts of the river channel.

Much of the levee building in California has been done as a part of the swamp-land reclamation work. The reclaimed land was of very great value for agricultural purposes, but its usefulness was greatly impaired by the constant and growing uncertainty and danger from flood waters. The usual practice had been to completely inclose in levees districts varying in size from a few acres to 50,000 acres each. The levee systems had grown intermittently and irregularly from small hand-built levees to large dikes 10 to 15 ft. or more in height. In the beginning little use was made of engineering skill or study,

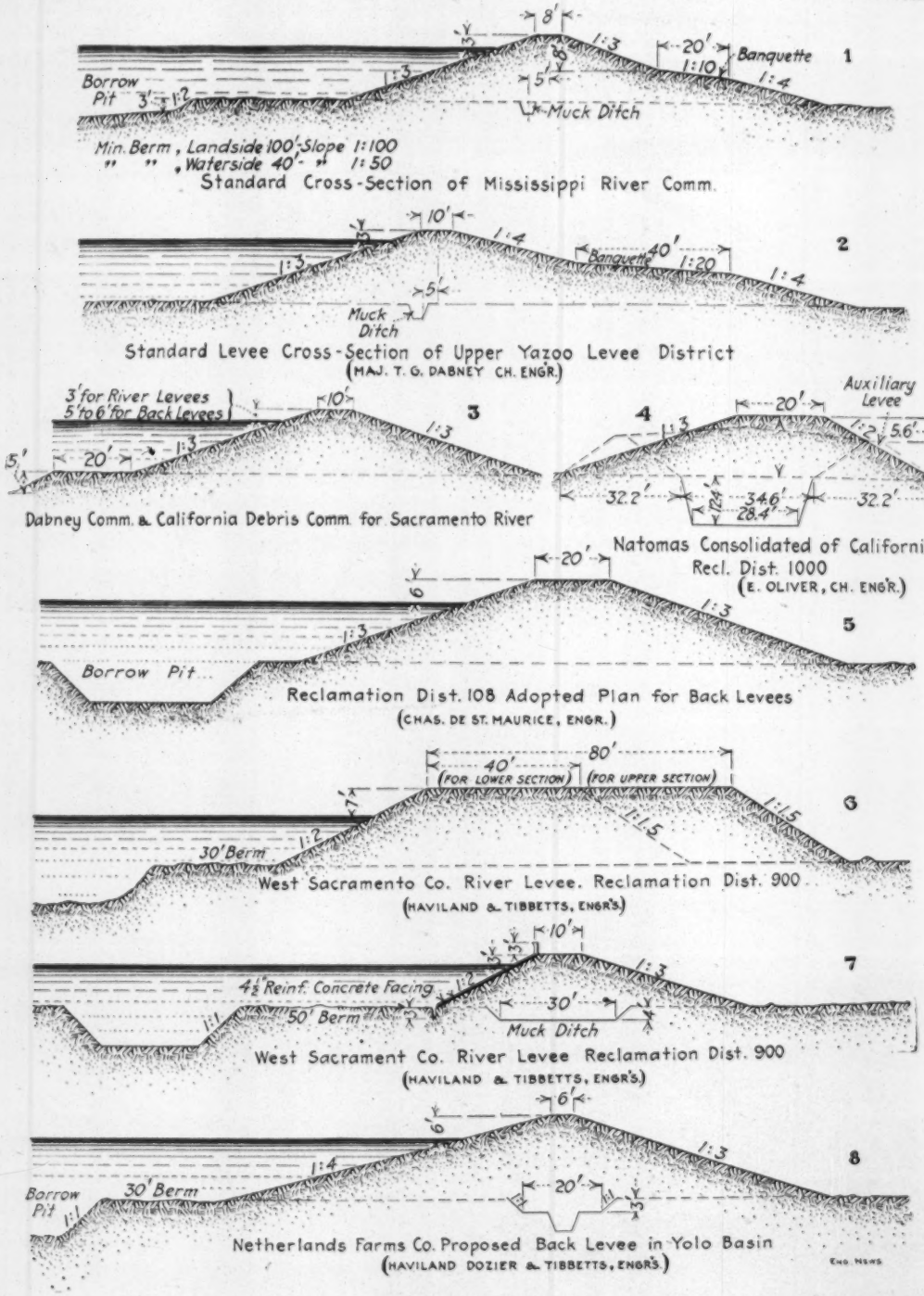

FIG. 6. COMPARATIVE CROSS-SECTIONS OF MISSISSIPPI AND SACRAMENTO RIVER LEVEES

and the machinery in use was of insufficient capacity to properly construct the necessary levees. The location, design and construction of most of the reclamation work were costly and inefficient.

The last decade has put an entirely different aspect upon this situation. Flood-control problems have been studied thoroughly and scientifically* and engineering talent has been concentrated on the large reclamation districts. Heavy dredging machinery has been developed further in some lines than anywhere else in the world.

The levee cross-sections have been gradually enlarged and improved and construction methods perfected, until it is doubtful if there are any larger or better levees anywhere. As compared with larger river systems, like the Mississippi for example, the maintenance of river levees is much easier in the Sacramento Valley, because of the briefer duration of the floods, which are hardly long enough to permit of the saturation of the levee sections.

On the other hand, the back levees located in the flood basins in many cases have a terrific exposure to wave

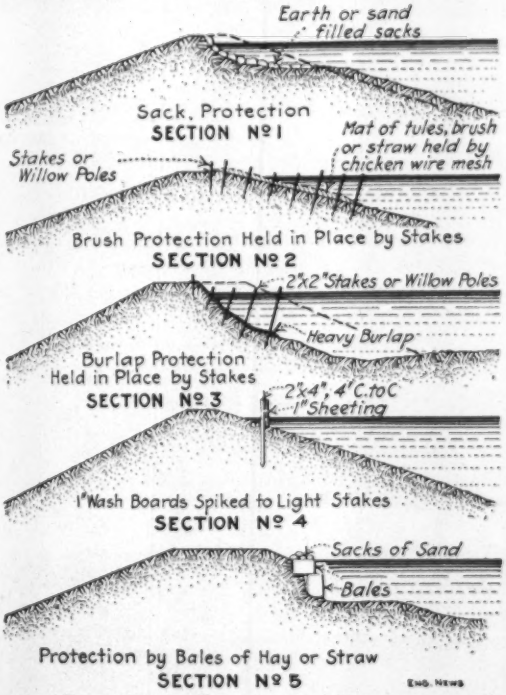
Earth or sand filled sacks
Sack Protection
SECTION Nº 1
Stakes or Willow Poles
Mat of tules, brush or straw held by chicken wire mesh
Brush Protection Held in Place by Stakes
SECTION Nº 2
2"x2" Stakes or Willow Poles
Heavy Burlap
Burlap Protection Held in Place by Stakes
SECTION Nº 3
2"x4", 4' C. to C.
1" Sheeting
1" Wash Boards Spiked to Light Stakes
SECTION Nº 4
Sacks of Sand
Bales
Protection by Bales of Hay or Straw
SECTION Nº 5
Eng. News

FIG. 7. METHODS OF PROTECTING SACRAMENTO RIVER LEVEES FROM WAVE WASH

wash during the winter storms. The growth of willows is generally encouraged on back levees, and climatic and other conditions are so favorable that where there is a proper berm, it is frequently possible to get a dense covering in a short time. In some cases, however, this method has been found inadequate and the back levees have been faced with concrete.

Many of the levees in the Sacramento Valley have been quickly destroyed by successive floods, and abandoned; but meanwhile new projects are constantly under way, so that it is impossible to estimate accurately the extent of land reclaimed or partially reclaimed at a given time.

*See Weather Bureau Bulletin 43, U. S. Department of Agriculture, entitled "The Rivers and Floods of the Sacramento and San Joaquin Watersheds," by Nathaniel R. Taylor, issued June 7, 1913; for very complete analysis of floods and river conditions.

The best estimates available at the present time indicate that the area of reclaimed land wholly in the flood basins and which could be said, under normal conditions, to be submerged with each moderate or large flood, is about 145,000 acres north of Sacramento, and about 110,000 acres south of Sacramento, making a total of 255,000 acres, or about 30% of the total of 800,000 acres.

Reclamation work in the Sacramento Valley is proceeding at present at a wholly unprecedented rate. Actual construction work is being actively pushed on tracts aggregating over 280,000 acres, or over one-third of the total flood area. All of these projects should be completed within the next three years. This means that the present period of say three years will see more land reclaimed and put under cultivation than the entire period preceding since the settlement of the valley.

Most of the projects under way, or proposed for immediate construction, are relatively large. Economical considerations make it more profitable to reclaim a large body of land than a small one. The recent organization of the State Reclamation Board and of the Sacramento-San Joaquin drainage district, embracing about 1¾ million acres and the comprehensive plan of the California Débris Commission insure future reclamation, following an orderly and legitimate plan.

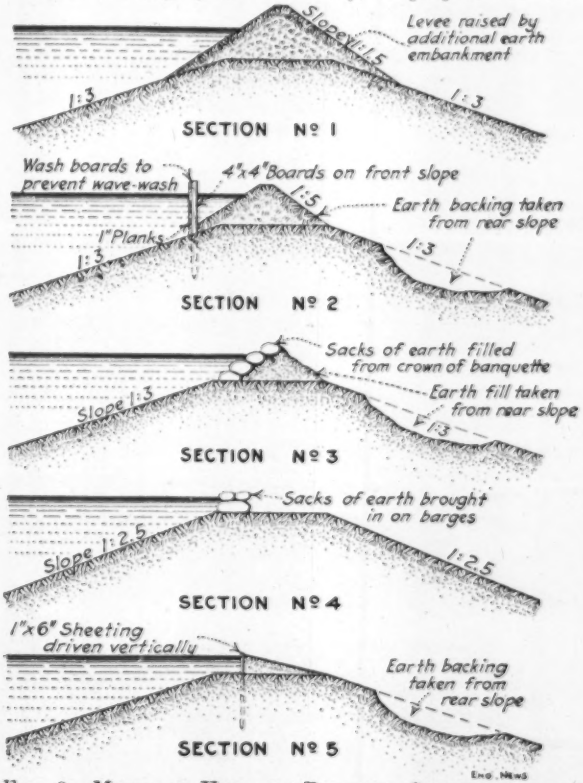

FIG. 8. METHODS USED TO PREVENT OVERTOPPING OF SACRAMENTO RIVER LEVEES DURING FLOODS

The large reclamation districts are employing the best engineering and executive talent available, and under a broad and far-reaching financial policy are insisting upon the most thorough and substantial construction work. The disastrous mistakes of the past are not being repeated. The reclaimed lands are being provided with transportation facilities and are being rapidly settled and put under intensive cultivation.

Progress on the Lower Ganges Bridge, Northern Bengal, India

By M. D. Brayshay*

One of the largest bridges in the world is now in course of construction across the Ganges River, at Sara, about 150 miles north of Calcutta, India. The project was described in Engineering News, Nov. 9, 1911, p. 553, when the construction work had just started. The progress during the past two years has been rather slow but sufficient work has been done to warrant the following progress report on the structure.

This bridge will carry the Eastern Bengal State Ry. across the river Ganges 120 miles from Calcutta. It is designed for two tracks of 5-ft. 6-in. gage and a 5-ft. footpath bracketted on the outside of the downstream girder. The estimated cost of the bridge and training works is £2,600,000 ($12,500,000) and, including the approaches, amounts to about £3,200,000 ($15,500,000). The Ganges here rises 31 ft. in the "monsoon" season and has a maximum discharge of 2,500,000 cu.ft. per sec. The river at the point of crossing runs almost due north and south through a sandy alluvial country which lies for the most part below the high-water level of the river. The river has frequently wandered widely from its present course and borings show sand to a great depth below the bed. Extensive protection works and very deep foundations are essential to keep the river in its channel and prevent failure by scouring of the foundations.

Protection Work—There is a guide bank along each side of the river extending 3000 ft. upstream and 1000 ft. downstream from the center line of the bridge and curving away from the river at the ends. There are two more guide banks—one three miles upstream on the left bank and one seven miles upstream on the right bank—to hold the river to its present course. These guide banks are of the "Bells bund" type and raised well above high-water level. They have a core of earth faced with pitching stone and a heavy pitched apron of stone designed for a scour of 100 ft.

Foundations—The piers are founded on wells sunk by open dredging to a depth of 150 and 160 ft. below lowest water level. The well is 63 ft. long and 37 ft. wide with semicircular ends and two dredging holes of 18-ft. 6-in. diameter. The well curbs are of steel, 15 ft. 7 in. deep, and weigh 140 tons each. For founding in water, the well curb is continued upward as a caisson, the height being adjustable up to a maximum of about 65 ft. The well curb and caisson are filled with concrete and above them the steining is built up of concrete blocks of about 7 tons each. After sinking the dredge holes are plugged top and bottom with concrete. The dredges used are of the Bell's type and of 100-cu.ft capacity. These dredges and the materials are handled by two 15-ton electric traveling cranes mounted on an overhead floating staging. An average rate of 3 ft. of building and sinking per day can be maintained with this plant.

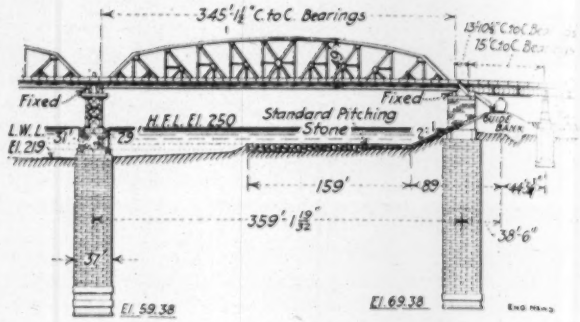

Fig. 1. Shore Span of the Lower Ganges Bridge, Sara, India

(There are 15 spans identical with this one and three plate-girder approach spans.)

Piers—Up to highest water level the piers are constructed of masonry and above that there is a steel trestle 34 ft. 6 in. high. The masonry consists of base and cap slabs of reinforced concrete and between these a core of brickwork and face of molded concrete blocks. The trestle was adopted to reduce the weight on the well foundations. Neglecting friction and buoyancy the unit pressure on the base of the well is about 13 tons per square foot.

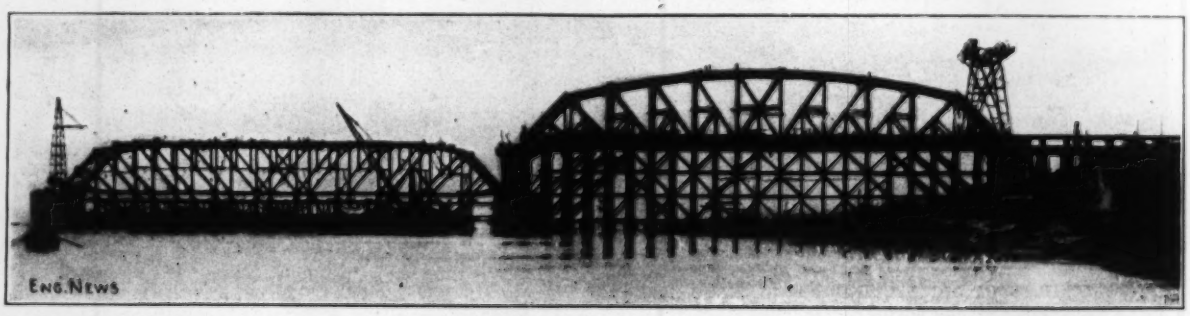

Fig. 2. Shore Span on Right; Service-Erecting Span on Left

Trusses—There are 15 through spans, 345 ft. 1½ in. c. to c. of bearings, and 359 ft. c. to c. of piers, and six plate-girder deck-approach spans 75 ft. long, the total length, abutment to abutment, being about 5890 ft.

Each main span is 52 ft. deep at the center and weighs 1250 tons. The clearance above water is 40 ft. at high-water level and 71 ft. at low-water level.

The erection in the water spans is to be done on a service span (336 ft. 5 in. c. to c. of bearings) which bears on the masonry pier and acts as a staging. This is shown in the view in Fig. 2. This service span can be floated from span to span on pontoons. Where there is insuf-

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sient water for the service span, timber staging is used, as in the shore span in Fig. 2. An electrically operated traveler (Fig. 4) lifts the pieces of the girder into position.

PLANT—Two power houses one on each side of the river, generate power for operating most of the plant on this bridge. The plant includes five steel overhead stagings each mounted on two pontoons and carrying two 15-ton traveling cranes, three 10-ton, 80-ft. radius floating Scotch derrick cranes, two erection travelers, two 10-ton, 53-ft. span travelers for handling concrete blocks, two mechanical excavators, capacity 80,000 cu.ft. each per diem, five stone crushers and a large number of concrete mixers, riveting machines and workshop plant.

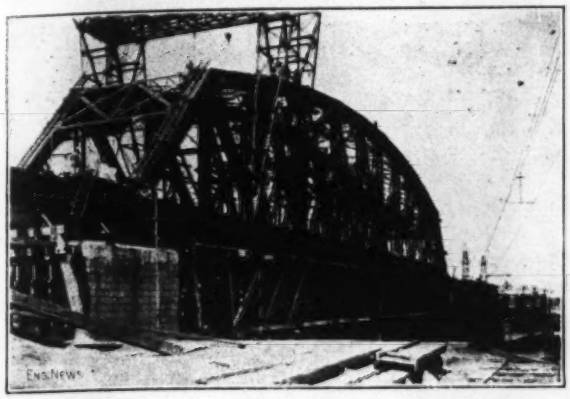

FIG. 3. LOOKING ACROSS GANGES RIVER FROM SHORE SPAN

PROGRESS OF THE CONSTRUCTION WORK—The "monsoon" season lasts from June to October and the working season is from November to May. Great care has to be taken to start nothing which cannot be carried to a condition of safety by the end of the season. Work on the guide and approach banks started in season 1910-1911 and they are now nearly completed. These works contain about 7,800,000 cu.ft. of earthwork and 900,000 cu. ft. of pitching stone. The stone has had to be carried by river and rail an average distance of about 150 miles.

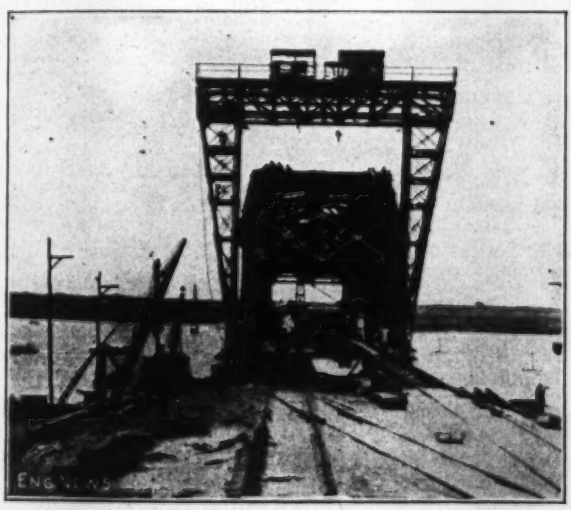

FIG. 4. STEEL TRAVELER USED IN ERECTION OF LOWER GANGES BRIDGE, INDIA

Owing to the strikes in England the work on the wells, piers and girders has been delayed considerably. Five wells were sunk in 1911-1912 and during the last season seven wells have been put down and one main span erected as well as the service span. The twelve wells completed represent some 220,000 tons of steelwork and masonry. It is believed they are the deepest foundations of their kind in the world. The two spans erected did not arrive from England until May, and the engineers in charge had a difficult problem to tackle in deciding whether to start erection at so late a date. The service span was "struck" on June 14 and the main span on June 17 on parallel drifts. The rivet holes were filled with 40% of parallel drifts and 60% of service bolts and the deflection was ¼-in. less than had been estimated for rivets. On June 15 the river started to rise rapidly and on June 18 it topped the piles of the stagings and the current was very strong.

The accompanying drawing (Fig. 1) shows the general elevation of the bridge and the photographs show the girders and stagings at different stages during erection.

The bridge is being built departmentally by the engineers of the Indian Public Works Department with R. R. Gales, M. Inst. C. E., M. Am. Soc. C. E., as Engineer-in-Chief. The steelwork is being supplied from England to drawings by the Consulting Engineers to the India Office (Messrs. Rendel, Palmer & Tritton).

NOTES FROM THE ENGINEERING SCHOOLS

IOWA STATE COLLEGE—At the last session of the State Legislature provision was made for two-year trade-school engineering courses in connection with the Engineering Department. These courses will be open to any young man with a common-school education. It is intended to give practical work in cement products, road-making, power generation and applied electricity to fit students for positions in the Iowa industries. Wood working, pattern making, forging and machine work are to form an important part of the instruction.

In addition, the college this year will carry on extension work in industrial subjects for apprentices and shopmen throughout the state. This work is now being organized by Prof. Kenneth E. Smith, formerly in charge of the technical extension work of the University of Wisconsin in Milwaukee. Evening classes in shop mathematics, mechanical drawing and shop practice, and fully organized part-time trade schools will be established in the industrial centers of the state.

A $100,000 locomotive- and automobile-testing laboratory, a $40,000 gas and steam laboratory and a $250,000 chemistry building are being built at Ames.

Dismissing Factory Employees Daily by Ringing the Fire Bell was suggested by Roland H. Leveridge, of the New Jersey Department of Labor in consultation with the Industrial Board of the New York State Department of Labor, which held its third public hearing upon the matter of factory safety Aug. 22. By this means the employees would become accustomed to the sound of the bell, thus lessening the likelihood of a panic in case of fire. It would also necessitate maintaining the alarms constantly in order.

The Problem of Finding the Proper Turbine Capacity to be Installed at a Water-Power Site

By Clemens Herschel*

The problem above stated arises at least once in the life history of every power plant.

Certain general rules have recently been promulgated by Hon. Franklin H. Lane, Secretary of the Interior, describing the policy of the Government as to the disposition of water rights on navigable rivers. The first of these rules reads as follows:

> The greater the development of horsepower, the lower the charge per horsepower to be made on the part of the government. This is intended to secure the full use of the stream.

Plainly this means that a large *specific,* or proportionate use of water at any mill site, is to be rewarded, in form of a less charge of water per cubic foot per second of turbine capacity or per horse-power produced hydraulically at that site.

This still leaves open the question: How shall this most expedient or proper turbine capacity at any site to be installed, be determined? The turbine capacity to be adopted must be sufficient, and yet prudent; large, but not excessive, or uneconomical. If made too small, too much water passes by unused. If made too large, some of the turbines and attached generators stand idle too long in an average year; and become food for the scrap heap, together with some of their appurtenant operating outfit.

The question how to determine the proper capacity thus primarily, or eventually, to be installed, is only now becoming recognized as a question to be settled by some sort of computation. Recent German writers have given us solutions that may be characterized, in the opinion of the writer, as "fearfully and wonderfully made." See, for example, an article in *Zeitschrift f.d.g. Turbinenwesen,* 1913, pp. 209, 230, 244 by Government engineer (retired), Dr. (of Engineering) Leiner. He calls his essay a "Procedure for the Determination of the Most Economical Part to Be Used of the Annual Water Supply; Size of the Reservoir; and Amount of Auxiliary Heat Power; Which Should Be Selected in the Construction of Water Power Plants."

As the learned author treats the subject, this includes a choice and determination: (a) Whether to use no water at all, or all the water that comes down the brook or river, during the year; or how much of the annual discharge of the stream is to be utilized; (b) whether to have no reservoirs, or reservoirs large enough to utilize the whole annual river flow; or what intermediate size of reservoir capacity is to be provided; (c) whether to have water power only, or to use auxiliary heat engines; and if any are to be used, to what extent.

This is certainly a comprehensive program. A similar program, and procedure under it, is set up in: "Die Wasser Kräfte," by A. Ludin, 1913, Julius Spri... Berlin. But we may venture to say, that no man ... has designed, built and operated water-power plants ... 20, 30 or more years, will tamely submit to pass ... the yoke of any such by averment all-conquering ma... matical procedure.

Much may be pardoned, no doubt, to able mathe... ticians, grappling with the problems presented by ... construction of water-power plants, in size wor... of the name, to be constructed for the first time in ... history of their nation; and before the nation has ... a condign experience in their construction and operati... But in this country there has been ample experience ... the subject; beginning in 1792, with the Society for ... Promotion of Useful Manufactures, at Paterson, N. J., ... and continuing through the upbuilding of Lowell, La... rence and Holyoke, Massachusetts, and all the other grea...

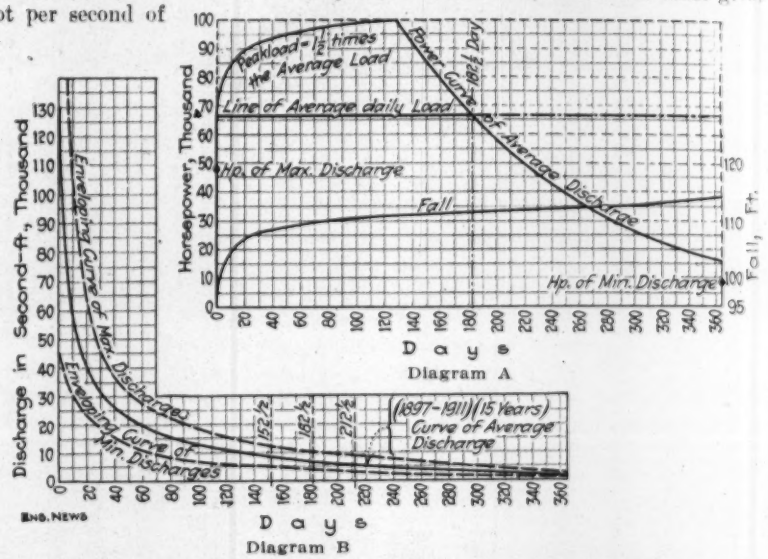

Diagram A

Diagram B

Diagram A. Curves Plotted from 15 Years' Stream-Flow Records of Potomac River at Aqueduct Dam, Washington, D. C.

Diagram B. Diagram for Determining Turbine Capacity for Installation at a Water-Power Site

water powers of New England and of the United States, ever since, inclusive of the marvelous advances in the art made since 1890.

We know that legal and practical considerations will settle, before the works are begun, the question of the size of supply reservoirs, or of mill ponds, to be constructed.

So long as the necessary use of lands in the construction or operation of works serving to retain or convey water is not by constitutional statutes in the states made a public use, so long will computations such as are above described, have no adequate justification within those states. Lands, and rights of way over them, must now be bought by private agreement, which practically and effectively limits the extent of their acquisition; not to mention the topographical limitations of each case.

Once thus remove from the computation the variable factor of size of reservoirs to be provided, and differently express the percentage of annual discharge of the stream to be utilized, and we come to a much simpler mode of treating the problem stated at the head of this article.

*Consulting Engineer, 2 Wall St., New York City.

The present writer has given a solution of this question in H. R. Doc. 1400, 62d Congress, 3d Session, in a Report made to Lt. Col. Wm. C. Langfitt, Corps of Engineers, U. S. A., dated Jan. 7, 1913, on the Great Falls water power of the Potomac River, near Washington, D. C., which may now briefly be described.

For this purpose p. 46 of Document 1400 may be quoted as follows:

ANALYSIS OF 15 YEARS' DAILY FLOW, 1897-1911, OF THE POTOMAC RIVER AT CHAIN BRIDGE, AND DETERMINATION OF THE ECONOMICAL CAPACITY OF THE RIVER, AT THAT POINT, TO GENERATE POWER.

So called hydrographs of river flow, as usually presented, form a most incongruous and uninstructive series of sawpoints, only to be called "curves" by a stretch of the imagination, or by a strained exercise of respect for mathematical terms and definitions. But when the several daily quantities portrayed in such a hydrograph have been arranged in the order of their size, placing the maximum quantity for any one day in the year as an ordinate over, say, the first day in the year, and so on down to the smallest daily quantity for that year as an ordinate over the three hundred and sixty-fifth or last day of the year, one obtains a very regular smooth curve, called a "duration curve," because from it may directly be read the duration during the year of any given river flow shown on the duration curve. The same data may also be presented in the form of a table, in which case it will be sufficiently accurate for present purposes to present only the quantities obtaining every tenth day in the serial arrangement that has just been described.

Such a table for the Potomac River at Aqueduct Dam, 1897-1911, is given in Document 1400; and a similar 20-year table showing the discharge of the Connecticut River at Holyoke, Massachusetts, is printed in "Transactions of the American Society of Civil Engineers," p. 31, 1907.

Diagram A (herewith reproduced), presents the same data as the table just referred to. Of especial interest is the quantity found to obtain on ordinate 182½, on the curve of average flow; in the present instance, about 6800 cu.ft. per second. The reason for this interest is found in the fact that experience on rivers used for water power in this country, sometimes extending for a century or more, has shown that this is the quantity which ordinarily (or by a consensus of opinion and experience) represents the proper and most efficient capacity of turbine and other water wheels that may be installed.

* * * * * * * * * * * *

Of course, there can be no hard and fast rule about a matter of this sort, and a variation of 30 days either side of the 182½-day ordinate spoken of is, in cases, perfectly admissible. In our case this would represent a range from 5400 to 8500 cu.ft. per second at Aqueduct Dam.

To enter upon a computation as to the exact amount of river flow it is proper to utilize would lead to so many necessary basic assumptions, founded on judgment, that the whole computation would in its results be of no more intrinsic value than a judgment exercised to begin with on the basis of general experience.

Guided by a judgment of this sort I have computed in what follows for a proper draft for power of 6800 cu.ft. per second at the Aqueduct Dam.

And that this is no extravagant or yet a niggardly figure is also shown by the discharges of the river, selected hitherto, by other engineers who have designed hydro-electric works for the utilization of the Potomac River at Great Falls.

Frizell, in 1894, planned to use 5000 second-feet.
Herschel, 1902, planned to use 5000 second-feet.
Sellers and Engstrom, 1903, not stated.
Shirreffs, 1904, planned to use 10,000 second-feet.
Kennedy, 1907, planned to use 7320 second-feet.

The last paragraph above quoted shows that the method here described of computing the economical capacity of the Potomac River, agrees, broadly speaking, with the consensus of opinion on the subject exercised by the half a dozen hydraulic engineers who have worked on it during the past 20 years.

With the data now easily procured, namely the flow of the river in form similar to Diagram *A*; an assumed turbine efficiency; a peak load equal to a certain multiple of the average load for the 24 hours; and falls acting at the mill site (varying with the river discharge); a table may be computed, or a diagram similar to Diagram *B* (herewith reproduced) portraying the tabular figures, may now be drawn, thus fully solving the question stated at the head of this article.

It will not be necessary to repeat here the arguments of Document 1400, against allowing the development of mill sites, in their ultimate stage of development, for less than the economical turbine or power capacity that may be determined by the method which has been illustrated. Such a course would be wholly wrong; because irretrievably and continuedly wasteful; and thus repugnant to all proper provision for the due conservation of the natural resources of the country.

*

The Opportunities of the Average Man in the Technical Field

By O. E. Huebner*

Three classes of trained men—mechanics, draftsmen and engineers—perform the greater portion of the practical work in the engineering field. Greater attention is given each year to the pay of engineers and through the labor unions still more attention to the pay of the mechanics. The draftsmen through their lack of organization are more or less at the mercy of their employers. From every direction come enticing statements of the glittering opportunities in the engineering field: the mechanic may become a draftsman; the draftsman may become an engineer, and the engineer may reach the pinnacle of his profession or may attain some high and lucrative position in the business world. The fact which is generally overlooked is that the great majority of men in these three divisions of the technical field cannot ever rise above the average, or at least sufficiently to gain access to a position in the field beyond.

It is the purpose of this article to show in a general way the financial value of the average technical man. This will be measured by the ratio of his possible yearly savings to his total first cost. The latter figure represents the total expenditure in preparing him to practice his trade or profession, and is estimated to be a constant value of $3000 to bring him to the age of fourteen plus

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EXPENDITURES AND RETURNS OF MECHANICS, DRAFTSMEN AND ENGINEERS

MECHANIC				DRAFTSMAN				ENGINEER			
Years	Progress	Investment	Return	Years	Progress	Investment	Return	Years	Progress	Investment	Return
to 14	First Cost	3000	....	to 14	First Cost	3000	....	to 14	First Cost	3000	....
17	Apprentice	....	500	17	High school	600	....	18	High school	800	....
				18	Tradeschool	600	....	22	College	2000	....
	First Cost Total	2500	....		Total First Cost	4200			Total First Cost	5800	....
Average Yearly Wage			1248			1332				1920	
Average Yearly Expense			1072			1172				1500	
Average Yearly Possible Saving			176			180				420	
Saving as per cent Investment			6.9			4.3				7.2	

the net cost of maintaining him until he enters the technical field.

The wages and salaries used in this article are the average prices paid for technical labor in the vicinity of New York City. The accompanying table shows in a clear and concise manner the expenditures and returns of mechanics, draftsmen and engineers. The average wage of the mechanic is assumed to be $24 a week; that of the draftsman $26 a week, and the salary of the engineer $160 per month. The average yearly expenses for each class of men have been very carefully figured and it is believed that the possible saving shown represents very fair values. But on the other hand the expenses are principally what might be termed of necessity. The many little luxuries so necessary to the smooth running of the complex human machinery are not possible of calculation. Nor are the expenditures approximated which contribute to various mental satisfactions. The figures represent "keeping body and soul together."

The tangible values of the investments of the mechanic and the engineer seem to be about equal, 7%. The dividends on the draftsman's investment fall considerably below the other two, due to the fact that it costs him $1700 to equip himself to earn $2 a week in excess of the wage of the mechanic.

A paying investment in which a man is personally interested should pay a dividend of at least 10%. The average of none of these three classes of men reaches such a value, and can, therefore, scarcely be considered as gilt-edge investments. All risk their lives, which means their investment—the mechanic and the engineer through the perils of the trade and profession, respectively, and the draftsman through his predisposition to certain diseases as a consequence to his unhealthy posture and to the confining nature of his work. In addition the latter must often be out upon the work in order to keep in intelligent touch with it, thus subjecting himself to many of the dangers incurred by the mechanic and the engineer.

Tabulations such as the one presented should be made public from time to time so that they may act as guides in the selection of trades or professions. They are also of value as stimuli or energizers to those who have lain mentally dormant too long. It shows such men what they were, what they are and what they very probably will be.

Lining an Irrigation Ditch with Hess Metallic Fluming

By Elbert M. Chandler*

The Burbank Co., of Burbank, Walla Walla Co., Wash., owning an irrigation project of 14,000 acres in the vicinity, constructed several miles of canal lining of different types during the last winter. One piece of work which is novel, and may prove to be of considerable interest to engineers, is about 3000 ft. of lining constructed of "Toncan" metal, in the form of 6 ft. 4½ in. semi-circle.

The portion of the canal to be lined was located on a steep, sandy, side hill, where an earth canal was not practicable because of the porous character of the sand and soil, and because it would have been difficult to maintain the canal banks owing to the light character of the material of which they would be composed and the exposure to strong winds. Concrete lining was not looked upon with favor for this particular work because of the great scarcity of suitable sand, gravel, and water; and the necessity for completing the work in a very short time. About three weeks' time was allowed between the date that the ground was thawed out and the commencement of the irrigation season. During that period of time, about two miles of concrete canal lining were being laid on another portion of the project, and owing to this straining the capacity of the equipment and organization available for that class of work, relief was sought in form that would make possible rapid construction on the ground and at the same time provide an impervious lining having a reasonably long life.

After considerable thought upon the subject, it was finally decided to purchase 3000 ft. of No. 120 Hess improved metallic flume, constructed of No. 22 gage (0.028 in.) toncan metal and held in shape by steel ribs and bands heavily coated with asphalt. Toncan metal is represented to be nearly chemically pure iron and supposedly free from corrosion. In any event, its life would probably be a great deal more than that of galvanized steel. Owing to the light and sandy nature of the soils in the vicinity of Burbank, most any lining, to be of maximum value, must be so built that the drifting soil

Fig. 1. General View, Showing Maximum Height to Which Backfill Should Be Carried

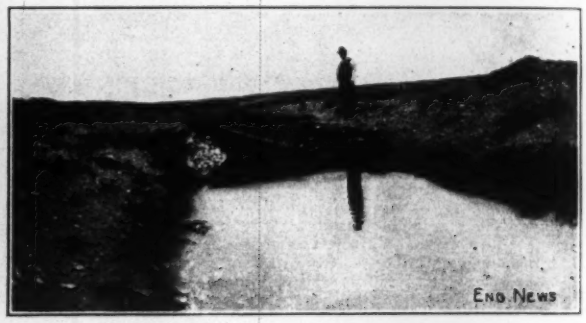

Fig. 2. Discharge End, Showing Riprapped Junction with Unlined Ditch

Views of Irrigation Ditch Lined with Hess Metallic Fluming, near Burbank, Wash.

*Manager and Chief Engineer, The Burbank Co., Burbank, Wash.

can be readily cleaned out. This would not be the case with this type of metal lining. Fortunately, in this case, the canal requiring improvement was located in such a manner as to be well exposed to the action of the wind and its line was approximately parallel to that of the prevailing direction of the wind. It was considered that the smooth interior of the Hess flume would permit the soil drift to pass through the flume without deposit by force of the wind, and during the irrigation season, the velocity of the water would be sufficient to carry away any deposit.

The lining was constructed by excavating the old canal bed to grade, setting line and grade stakes 25 ft. apart, erecting the steel in the same manner as in the erection of the ordinary steel flume, and correcting for alignment and grade as back-filled by hand with shovels.

In constructing a metallic flume, it is ordinarily highly essential to remove all grit and sand from the joints before placing them together in order to prevent leaks. Owing to the close proximity of the lining to the ground surface during erection in this case, this would have been difficult to perform, and the condition was overcome by applying a heavy asphaltic paint to the joints.

On account of virtually erecting the lining in a trench and the consequent inconvenience of working about it, it was not possible to make the same measure of speed as in ordinary steel-flume erection. The best progress made in this work was 320 ft., with a crew of one foreman and seven men in ten hours. This did not include the excavation, but did include the erection and cinching of the sheets, the assembling of material for handy use, painting the joints, boring the holes in the 4x4-in.x8-ft. fir carriers employed, and back-filling sufficiently to permit of the flume being temporarily sustained on grade and line.

The cost of this work was as follows:

	Per lin.ft.
Steel flume, f.o.b. Burbank	$1.30
Timber carriers f.o.b. Burbank	0.07
Hauling flume and carriers	0.03
Excavation and back-filling	0.06
Erection	0.13
Engineering and miscellaneous	0.03
Total cost per foot	$1.62

The cost of labor was $2.50 per man per ten-hour day; foreman, $4 per day; team and driver, $4.50 per day. The material was hauled six miles over poor roads.

Although the lining has now been in use for several months, it is too early to predict what life it may have, but it is now serving its purpose in a most satisfactory manner, and gives promise of having been well chosen to meet the particular requirements. Its high cost relative to that of concrete lining is such that it would hardly be used generally, but under circumstances where concrete lining cannot readily be constructed at a reasonable cost, it may be used often to advantage.

Messrs. Sanderson & Porter, New York and San Francisco, are consulting engineers for the Burbank Company, and are represented on the ground by the writer as manager and chief engineer of the company. The Hess Flume Co., of Denver, Colo., furnished the steel flume.

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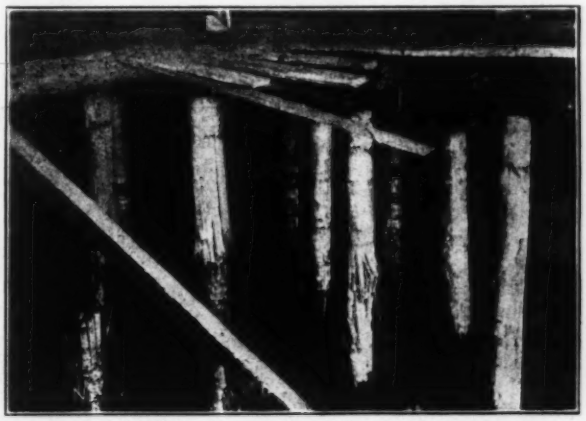
A Tool Used to Fight Fires in the California forests combines a rake, spade and hoe. It is compact, so that it can be carried on horseback, and weighs less than 5½ lb.—Forest Notes, U. S. Forest Service.

An Interesting Pile Failure

By John W. Cunningham*

On June 9, 1913, a portion of the floor of a covered timber dock known as Montgomery Dock No. 1, at Portland, Ore., collapsed, dropping about 4000 bushels of grain into the Willamette River, which was at a high stage at the time, and within 4 ft. of dock level. The river has since fallen, exposing the greater portion of the piling, and showing very clearly the nature of the failure.

The dock was a typical pile structure, and having been built in 1882, had outlived the usual period of usefulness for this form of construction. The piles were spaced 8 ft. by 8-ft. 3-in. centers, with 12x12-in. caps, 6x12-in. joists 4 ft. c. to c., and 3-in. planking. The load on the floor at the time of failure was wheat, sacked for export, the sacks being piled eight high and as closely as possible, giving a computed load of about 300 lb. per sq.ft., or 10 tons per pile.



Worn-Out Piles under a Portland, Ore., Dock

The accompanying halftone, from a photograph, shows that the first failure was in the piles, from simple decay during their 31 years of service. About an inch of the sap wood of each pile, the portion which had dried out rapidly after each annual flood of the river, was fairly sound; while the heart, which remained saturated, had become thoroughly rotten. It appears that the failure began by crushing the spongy core, the outer layers splitting and bending, but holding the upper and lower portions in line. Final collapse occurred when the slivers were broken, and the upper part easily sheared off to one side.

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The Production of Building Materials in 1912 in British Columbia, Can., according to the annual report of the Minister of Mines, for the Province, was as follows: cement, $800,000; lime and limestone, $151,250; building stone, $675,000; crushed rock, $315,000; sand and gravel, $505,310; red brick, $354,500. The only cement manufacturer in the province is the Vancouver Portland Cement Co., with works at Tod inlet, about 12 mi. from Victoria. The capacity of these works is over 2000 bbl. per day and during the past year 520,000 bbl. were manufactured. The raw materials, limestone and clay are obtained on the property of the company adjacent to the works. The Portland Cement & Construction Co. has been erecting a large plant in the vicinity, but production has not yet been begun. Concrete construction has become so extensive on the coast that the production of the material is commanding ever increasing attention.

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A Landing Cableway, Nome, Alaska

The accompanying two views show in operation a cableway which is used to transport both passengers and freight from steamships to the shore at Nome, Alaska. The view on the left shows a skipload of passengers being brought in from the steamer lying in the offing. Fig. 2 on the right shows a section of pipe being landed from the lighter in the near background which in turn has been brought in from the steamer just visible behind the pipe. The latter view was taken on June 22 and well illustrates the lateness of the season in this far northern seaport.

The cableway was built some years ago for the John J. Sesnon Co., of Nome, Alaska. The company at the time operated steamers from Seattle and San Francisco to Alaskan ports and owned warehouses and a wharf on the beach at Nome. The beach slopes very gradually so that vessels have to lie several miles off at sea and land their passengers by small boats or lighters. In onshore winds the sea is very rough and in the winter the waters freeze solid clear to the bottom for a long distance out. In order to provide a safe and continuously available method of landing, the company erected the cableway, part of which is shown in the views.

This cableway spans between the sea tower illustrated and a shore tower not shown in the views. The shore or head tower is 125 ft. high and the sea tower is 85 ft. high, the span between them being 1400 ft. A 2½-in. cable is used and the capacity of the cableway as designed is 10 tons.

The sea tower is built on a concrete base, which, in turn, is footed on timber piles driven in about 10 ft. of water. The concrete section is in plan a hollow square with the side towards the shore omitted. The off-shore face of the square is 48 ft. long and 10 ft. thick with corners thickened up to 12 ft. The side faces are 36 ft. in length, 3 ft. thick and have corners 8 ft. square. The tower is of timber. The sills and posts are 12x12-in. in section while the bracing is made of 3x12- and 3x12-in. timbers. The shore tower is of similar construction.

The tower and the cableways were built by the Roeblerwood Manufacturing Co., of New York City. This company has furnished several similar cableways for freight landings to various ports of the world, notably the harbor of Manaos, Brazil, and Hilo, Hawaiian Islands.

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A Water-Purification Plant for East Liverpool, Ohio, has been ordered by the State Board of Health, after a complaint had been made by the local Board of Health and a public hearing had been held, all as required by law. The city authorities responsible for the water-supply made no objection to the proposed order. The plant must be installed by Jan. 1, 1915, and its character must be satisfactory to the State Board of Health.

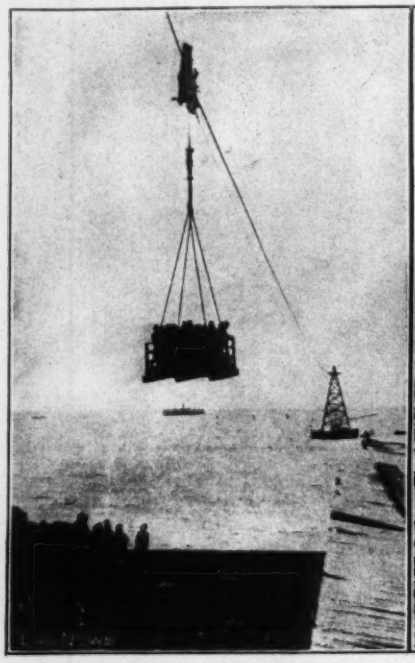


FIG. 1. LANDING PASSENGERS BY CABLEWAY FROM STEAMER AT NOME

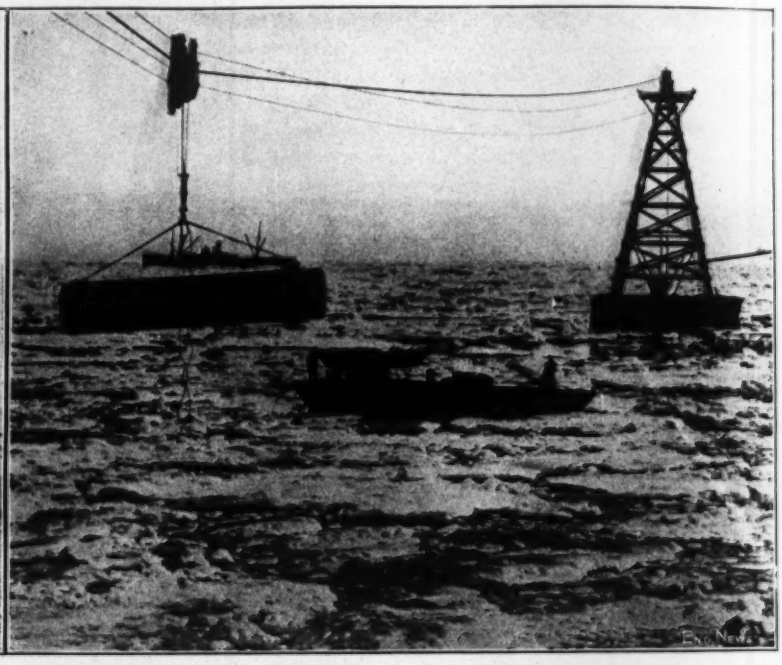


FIG. 2. LANDING A PIECE OF PIPE OVER A FROZEN SEA

Car Washing vs. Paint Preservation—M. B. Smith, Chief Chemist, Detroit United Ry., in "Electric Railway Journal," Aug. 9, 1913, gives results of an investigation of the methods employed by the company in washing the cars. The following recommendations are offered which apply generally to the washing of any painted surface: (1) the maximum temperature for wash water should not exceed 80 deg. F., and should be regulated by thermometer; (2) the use of too much soap may be avoided by supplying the car-washing stations with a stock soap-solution of a strength equivalent to 1.5 oz. per gal. of water; (3) rinsing must follow the soaping immediately to avoid attack by the soap on the car surfaces; (4) soda ash or other equivalent alkali must not be used as it removes the paint; (5) cars must be wet down over the entire surface at least twice to assure the softening of the accumulated mud and sandy particles.

In an article by E. W. Holst, Superintendent of Equipment, Bay State Street Ry. Co., Boston, Mass., in "Aera," the monthly publication of the American Electric Railway Association, it is claimed that a new car properly painted, constructed of first-quality material and re-shopped every 12 months should give at least five years' service before repainting becomes necessary. Half of the high maintenance cost is claimed due to decay and corrosion of wood and steel caused by exposure and lack of attention.

EDITORIALS

It has been stated before that most accidents are the fruits of carelessness. A man was killed on August 14 while uncoupling two freight cars in the yards of the Louisville & Nashville R.R., Birmingham, Ala. While he was *between* the two cars a switch engine backed into them. It was not a remarkable accident; in fact, its every-day aspect is its chief characteristic; but it represents a class of casualties which should be outgrown. Any man who places himself between two cars which are standing on the tracks of a busy railroad yard, invites serious injury or death. In the case of the present accident he does more: this victim was a yard foreman and, according to press dispatches, he was directing a night switching crew. If such a man jeopardizes his life in the presence of his men, what can be expected of the latter?

We are reminded by this accident of the recent safety car exhibit of the New York Central Lines (noted in our issue of Aug. 14). The walls of the car were lined with many photographs depicting the right and the wrong method (as regards safety) for performing certain operations in railroad yards; and the correct manner of coupling and uncoupling cars was one of the operations considered. The work of educating employees in safety methods should be taken up by all the railroads. The expense involved would be trifling compared with the advantages accruing from a higher standard of employee. The man who is careful of his body, which is his most valuable possession, may reasonably be expected also to be careful of the property of his employer.

The Control of Municipal Utility Rates

A brief summary was given in last week's issue of the main provisions of engineering interest in the very detailed public-service law recently put into effect in Pennsylvania. The law gives the regulating commission broad powers over public utilities except as they may be owned by municipalities. In these cases the supervision is confined practically to passing on the necessity of a new project and to requiring uniform systems of accounting and the maintenance of ample depreciation reserves.

The hesitancy with which legislators, bold in their protestations about the need of controlling the utility corporations, take up the same wise control of similar municipal services is lamentable. Oftentimes, the need of commission regulation of municipal utilities is more pronounced than in the case of private corporate services, though the need usually arises from the very different causes of incompetency, inexperience, irresponsibility and politics.

With the utility corporations the real directors are largely those who have put their own funds into the business. Most such utilities are organized with some 40 to 70% of the capital borrowed, so that the managers are spending their own money and taking good care (perhaps too good care) in most cases to preserve their investment and earnings. With the municipal utility, the capital is all borrowed and the managers are not subject to the constant restraint of personal participation.

If municipal ownership were as general as corporate control among all the utilities, perhaps its illogical and unfair rate systems would be more widely recognized. The greatest cause for complaint arises from the prices which the municipality pays for its own official services. For instance, in the case of an electric plant the street-lighting appropriations may be boosted to make cheap current for private customers. Sometimes the reverse may be true, but frequently no one in authority knows whether the charge to the municipality is really above or below the cost of service. In the case of the waterworks the burden of fire-hydrant service may all rest on the consumers and not at all on the tax payers.

So far as we recall, Wisconsin is the only state in which the legislators have had the courage to impose the same burden on all utilities alike and to ride the storm which the seekers of political privilege might raise; but the light of the Wisconsin results is not to be hid under a bushel, and it is only a question of time before the other States come to see the wisdom of such action.

A Second Defeat for State Aid for Municipal Sewage-Treatment Works in Pennsylvania

The proposition to give state aid to municipalities ordered to build sewage-treatment works has again met defeat in Pennsylvania. This time a bill to that end actually passed the legislature, carrying with it an appropriation of $1,000,000 ($500,000 a year for two years), but the bill was vetoed by the Governor.

The bill as passed provided that whenever any municipality should find it necessary under the laws of the state to cease discharging [untreated] sewage into any of the waters of the state, and when plans for sewage treatment had been submitted to and approved by the State Commissioner of Health, then the governing body of the municipality might petition the auditor general for its *pro rata* share of any funds available for state aid, the distribution to be based on the relation of the cost of the particular works to the aggregate cost of all works receiving aid within the appropriation period. During construction of any state-aided sewage-disposal works the Commissioner of Health "through his engineers and assistants" would have had the right, had the bill become a law, "to direct and supervise the work and material used in order that the same may be up to and in accordance with the plans and specifications approved by him as required by this act." The cost of this supervision would have been paid from the yearly appropriation of the State Department of Health, but would have been

collected from the amount due the municipality and returned to the department.

We understand that the bill passed the legislature without opposition; that the State Department of Health remained neutral on the subject; and that the governor's reason for vetoing it was lack of funds—the legislature having voted appropriations far in excess of the legal limit, leaving the Governor to pick and choose what items he would kill to bring the total within the law.

There is something to be said both for and against state aid for municipal sewage disposal, but for the present the weight of the argument seems to be against the proposition. It is obvious to anyone who has noted the activity of the Pennsylvania Department of Health in issuing orders to file plans for sewage treatment that if the orders were really to be enforced then $500,000 a year of state aid, distributed *pro rata* according to the cost of the various works, would be a mere drop in the bucket. Even that drop could not be provided this year through lack of funds. If the demand for state aid becomes sufficiently great hereafter then presumably some other appropriations will be sacrificed to meet the demand, but before such sacrifices are made there is likely to be serious questioning as to whether state aid is after all wise and also whether Pennsylvania is not going too fast in demanding plans for sewage treatment.

As to the latter point, it is true that comparatively few of the demands for plans have been followed by an insistence that the plans presented and approved be executed. This leads to the question, Why demand plans if they are not to be carried out? It is, of course, highly desirable to have the sewage-collecting system and the outfall sewer so designed that sewage-treatment works may be subsequently installed at a minimum of expense, but beyond this the chances are ten to one that plans for sewage treatment made this year but left unexecuted will be largely if not wholly remade five or ten years hence. Provision for the future is admirable but it should be kept within reasonable bounds. If municipalities are required to make plans for sewage treatment too far in advance of their execution or without due regard for both sanitary needs and financial capacity public opinion will be aroused and will not stop short with efforts to secure state aid.

Establishing a Uniform City Datum by Precise Level Methods

The work which the City of New York has recently completed in establishing a precise level net embracing the entire area of the city is worthy of the attention of every city engineer. Aside from the marking of street lines with sufficient care and accuracy to prevent encroachments, which in business streets are expensive to remove, there is perhaps no part of city survey work more important and more appreciated by private engineers as well as municipal employees than a really accurate and systematic net of benchmarks, referred to some one definite datum plane.

Six different datum planes have been in use by regular departments or bureaus of the New York City government, three different datums have been used by special boards in charge of public work and at least two different datums by private authorities in charge of semi-public improvements. Very probably a similar situation exists in nearly every large city. In such cases difficulties must always arise not only from the multiplicity of datums but from a lack of knowledge of the precise relation or difference in elevation between them. Costly errors are almost sure to result and an immense amount of otherwise useless computation must be gone through prior to starting any comprehensive engineering construction, or an entirely new system of levels and a new datum plane must be established.

No engineer will question the usefulness and value of a systematic level net when it is possible to obtain one. How then is it to be established? Ordinary leveling with Y-levels, when carefully done, gives fairly good results—running duplicate lines, nearly equal back and foresights, checking on the starting benchmark, and adjusting the elevations by distributing the errors of closed circuits give what some engineers term precise results. In the sense the term is used in geodetic work, however, precise results cannot be obtained with an ordinary Y-level, set up in the sunlight, and using a target rod and promiscuous turning points. Really precise results, that is, where the limit of error is less than 4.0 mm. K (K is the length of level line in kilometers), cannot be systematically obtained except with a level designed for precise work and by the use of special methods.

With the exception of a few of the larger cities (notably Baltimore and St. Louis), hitherto practically all such systematic precise leveling has been in charge of the U. S. Coast & Geodetic Survey or Geological Survey officers, or by special engineers of a few great railway companies—by men who have had a careful preliminary training in geodetic surveying. The average city engineer has been reluctant to try precise level methods, either from a feeling that such methods were an unnecessary refinement for his purposes, or from a mistaken notion that the field and office work is unusually laborious and difficult for inexperienced employees.

However, it must be borne in mind that most of the precise level nets established by the government engineers have been made by different parties and have covered very large areas, and consequently have involved a greater amount of adjustment computations than such work as has been recently done by the City of New York. How easy it is to get highly satisfactory results with a party not previously trained in this class of work, provided sufficient care and good judgment are used, will be appreciated by every reader of Mr. Koop's paper before the Municipal Engineers of the City of New York, abstracted elsewhere in this issue. That such accurate results are desirable when not achieved at excessive cost or by unusual skill will be readily admitted; and the New York City work offers an excellent example of how it can be done.

The work that the society, the Municipal Engineers of the City of New York, has accomplished in bringing about this precise level survey is highly commendable. This was one of the earliest activities of the society, and so far back as 1906 a committee was at work compiling a list of the different datum planes, their accepted elevations and their differences in elevation. The society consistently urged the attention of the proper authorities to the need of establishing a precise level net and in the end has been the indirect means of obtaining it.

While this particular society is composed wholly of men in the municipal service, and hence peculiarly inter-

ested in having a reliable system of benchmarks, every other local engineering society could, to a greater or less extent, do a vast amount of good and utilize a great deal of surplus energy in a perfectly legitimate field by aiding, both directly and indirectly, to systematize and simplify the city survey work of their communities.

The Diminishing Timber Supply and Municipal Forests

More than four years have elapsed since President Roosevelt's Conservation Commission submitted its report. Before that time and since then, public attention has been both spasmodically and periodically riveted on the diminishing timber supply and the need for conservation of forests. Some good work in forest conservation has been accomplished both by public authorities and by large private lumber companies, but in large degree the same old reckless methods of lumbering still prevail, the annual fire loss through carelessness and makeshift construction is still appalling, and today we are constantly getting nearer the end of the existing timber supply without having made much progress toward postponing its apparently inevitable end.

So long as the price of common lumber does not make any extraordinary advance and a sufficient supply is available for present needs, it is, and will be, practically impossible to impress upon the everyday user of lumber that the time is inevitably coming when he must either find a substitute or pay high for forest products. Engineers have indeed already come to a more or less full realization of the scarcity of timber suitable for many kinds of construction; and concrete, tile, steel, fiber and other materials are now able to compete with timber for these purposes. Yet for many purposes lumber still holds, and in all probability will continue to hold, its own. According to the United States Census Bureau, over one-half the lumber cut still goes into frame buildings and rough construction, about one-tenth into boxes, one-twentieth into car construction, and the remaining one-quarter into other uses. For many of these purposes wood will continue to have more and more competitors, which will tend to neutralize the increasing demand for lumber; and hence an extraordinarily rapid rise in prices is not likely to come until the timber supply in large privately owned stands is practically exhausted.

According to estimates of the United States Bureau of Corporations, the present stand of merchantable timber in this country will not last 60 years at the present rate of consumption. The consumption has remained practically constant for six years and is not likely to be materially reduced so long as common lumber does not reach prohibitive prices; which is not probable while a large portion of the available standing timber is owned by individuals or private corporations, who under the present system of taxation, can neither afford to keep the capital invested indefinitely or to run the risk of having their standing timber destroyed by forest fires by holding the forests in reserve.

The present rate of forest growth is only one-third of the rate of consumption. Since 1850 the center of the lumber industry has moved steadily westward, from New York to Pennsylvania, to Michigan, to Wisconsin, to Minnesota, to Washington. There can be no further westward movement; the shifting of the centers of production in the last 60 years shows all too graphically the change in timber resources.

In view of these facts the proposal of the city of Seattle to create a municipal forest of nearly 140 sq.mi., which will not only furnish protection to the water-supply of the Cedar Lake drainage area, but also become a valuable asset and a source of revenue, will be regarded with interest by all true conservationists, not because of the novel y of the scheme, for many other American cities have already forested their water-supply watersheds, but because of the scope of the project and the probability of a large profit from the sale of forest products.

While few cities own so large or so favorably situated a watershed as that of the city water-supply of Seattle, there are in the vicinity of the water-supply reservoirs of many cities, towns and communities thousands of acres of denuded land, now practically useless, which could be converted into real public assets by coöperative effort.

There is nothing new or novel in municipal forests. They have proved to be important sources of revenue to European cities. According to a report of the American Civic Association, in the single province of Baden, Germany, out of 1564 towns or communities, 1350 own their own forests; and in addition, 287 other public corporations, such as schools, churches and hospitals, own forest lands. The city of Baden alone, in 1909, owned 10,576 acres, from which it had, up to that time, derived a net revenue of $66,079. Massachusetts, and possibly other states in this country, have laws authorizing forest culture by townships.

In many states (including Washington), however, private forest lands are still taxed under the general property tax, a method long abandoned by European nations, instead of by a small annual tax (or none at all) on the land itself, exclusive of the value of the timber, and a tax up the timber when cut. Under the general property system of taxation individuals or private corporations will never invest large sums in reforestation, or take up forest culture as a commercial proposition. Such a system of taxation also works to prevent the perpetuation of present stands—a point clearly brought out in the Seattle investigation.

In the Seattle report it is estimated that the cost of growing timber to private parties at the end of the 70-year period would be nearly double that to the municipality, accounting the initial cost of the land at $2.50 per acre and taxes at 1% of real value (the present rate of taxation in Washington). Private corporations must also pay a higher rate of interest on invested capital and run a greater risk from forest fires, since they are not in a position to establish so efficient and complete a patrol system or to enforce the laws against careless campers and hunters.

It will take a long time to prove the truth of the prospects held out in the Seattle report on the reforestation of the Cedar River watershed, but it is to be hoped that both it and other cities will have faith enough in the future of forestry in the United States to embark on similar enterprises wherever local conditions seem to warrant it; for the time is surely coming when good timber of many desirable kinds will be very scarce and very high in price. Meanwhile the value of forests as public parks and recreation grounds furnishes an addi-

tional argument in their favor, and there are still many who believe that forests conserve water and soil, to say nothing of those who think, as the report of the Conservation Commission stated, that forests "abate the winds and give protection from excessive heat and cold."

An Ambitious River Regulation Plan

The widespread damage done by the recent Spring floods in the Middle Western States, and the subsequent agitation for a centralized supervision of the flood plains of the United States, has borne fruit in a bill introduced in the Senate of the United States by Senator Newlands, of Nevada, on July 14 of this year. The bill was read twice and then ordered to lie on the table. While it is extremely improbable that the press of business involved in the currency and tariff bills will allow any further consideration of the Newlands bill at this session, the bill is very interesting as an indication of proposed legislative action.

The title of the bill reads as follows:

A Bill—To create a Waterways Commission and a Board of River Regulation to promote interstate commerce by the development and improvement of the rivers and waterways and water resources of the United States and the coördination of and coöperation between rail and water routes, and by providing a fund for the regulation and control of the flow of rivers and for the maintenance at all seasons of a navigable stage of water in waterways and for the connection of rivers and waterways with the Great Lakes and with each other, and as a means to that end to provide for flood prevention and protection and for water storage and for the beneficial use of flood waters for irrigation and water power and for the conservation and use of water in agriculture and for the protection of watersheds from denudation and erosion and from forest fires and for the coöperation in such work of Government services and bureaus with each other and with States, municipalities, and other local agencies.

The bill first provides that the sum of $60,000,000 be appropriated annually for each of the ten years following July, 1913, to be expended in a so called River Regulation Fund, which, in the words of the bill,

is to be used to promote interstate commerce by the development and improvement of the rivers and waterways of the United States and their connections with the Great Lakes and with each other, and by the coördination of the coöperation between rail and water routes and transportation, and the establishment and maintenance of adequate terminal and transfer facilities and systems and their maintenance, improvement, and protection, and by the making of examinations and surveys and by the construction of engineering and other works and projects for the regulation and control of the flow of rivers and their tributaries and source streams, and the standardization of such flow, and by the maintenance of navigable stages of water at all seasons of the year in the waterways of the United States, and by preventing silt and sedimentary material from being carried into, and deposited in, waterways, channels and harbors, and by the conservation, development, and utilization of the water resources of the United States, and by flood prevention and protection, through the establishment, construction, and maintenance of natural and artificial reservoirs for water storage and control, and the protection of watersheds from denudation, erosion and surface-wash and from forest fires, and the maintenance and extension of woodland and other protective cover thereon, and the reclamation of swamp and overflow lands and arid lands and the building of drainage and irrigation works in order that the flow of rivers shall be regulated and controlled not only through the use of flood waters for irrigation on the upper tributaries, but also through controlling them in fixed and established channels, in the lower valleys and plains, and by doing all things necessary to provide for any and all beneficial uses of water that will contribute to its conservation or storage in the ground or in surface reservoirs as an aid to the regulation or control of the flow of rivers, and by acquiring, holding, using, and transferring lands and any other property that may be needed for the aforesaid purposes . . . and by securing the coöperation therein of States, municipalities, and other local agencies, as hereinafter set forth, and for the payment of all expenditures provided for in this Act; the ultimate purpose of this Act being the maintenance at all times of a navigable stage of water in all inland waterways, and flood prevention and protection, and river regulation and the control of the volume of water forming the stage of the river from its sources, so as to standardize the river flow, as contradistinguished from the supplemental to channel improvement as heretofore undertaken and provided for under the various Acts commonly known as river and harbor Acts.

For the administration of this fund, there is to be created a Waterways Commission, consisting of the President of the United States, who shall be Chairman of the Commission with the power of veto, the Secretary of War, the Secretary of the Interior, the Secretary of Agriculture and the Chairman of the Board of River Regulation, whose duties are noted further along in the bill; the Chairman of the Interstate Commerce Commission and the Chairman of the Panama Canal Commission are to be ex-officio advisory members of the Waterways Commission. In addition to the Waterways Commission, a Board is to be created, known as the Board of River Regulation, consisting of the Chief of Engineers of the United States Army, the Chairman of the Mississippi River Commission, the Director of the United States Geological Survey, the Director of the Reclamation Service, the Chief of the Bureau of Plant Industry of the Department of Agriculture, the Secretary of the Smithsonian Institute, one civil engineer, one sanitary engineer, one hydro-electric engineer, one expert in transportation, and a chairman of the Board. The last five are to be appointed by the President, and are to receive an annual salary of $10,000, except the Chairman, who is to receive $12,000. The remainder of the Board are ex-officio members. The five members of the Board appointed by the President constitute an executive committee, which is to have a general supervision of the operations of the Board, under rules and regulations to be established by the Waterways Commission. The Board is to prepare all plans and estimates for the expenditure of the money, and these estimates are to be approved by the Waterways Commission.

A section of the bill provides that wherever possible the Board of River Regulation shall encourage the cooperation of states, municipalities, public and quasi-public corporations, towns, counties, districts, communities, persons and associations, in the carrying out of the purposes and objects of the Act, and shall in every case endeavor to secure financial coöperation of these agencies, to an extent at least equal in amount to the sum expended by the United States. The authority is vested in the Board to negotiate the arrangements for the apportionment of the work, cost and benefit between these various agencies and the United States. It is a further duty of the Board to coördinate in the various scientific and constructive bureaus of the United States with each other and with the representatives of states, municipalities, etc., carrying out the purposes of the Act, and the Board is to have authority to call upon any other Federal department or bureau for help, the Board being authorized to defray the expense of such investigation.

The wide duties of the Board are embraced in the following paragraphs:

The Board of River Regulation shall develop, formulate, prepare, consider and determine upon comprehensive plans for the conservation, use, and development of the water and forest resources of the United States in such manner as will best regulate the flow of rivers and their tributaries and source streams, and the stage of water in inland waterways,

and the confinement of all rivers and waterways at all times within fixed and established channels, and embracing, with that object, the construction of levees and revetments and all works necessary for the fixation of channels and flood protection, drainage, and the reclamation of swamp and overflow lands; water storage in natural and artificial reservoirs; the beneficial use of waters for irrigation and for all domestic, municipal, and industrial purposes; the maintenance and development of underground water-supplies and the storage of water in the ground and in irrigated lands and underground reservoirs; the enlargement of the areas and raising of the levels of the ground waters; the construction of flood-water canals, by-passes, and restraining dams; the control and regulation of drainage and the replenishment of streams by return seepage; the perpetuation of forests and maintenance of woodland cover as sources of stream flow; the prevention of denudation and erosion; the protection of channels and harbors from eroded soil materials; the clarification of streams; the utilization of water power; the prevention of the pollution of streams and rivers; the sanitary disposal of sewage and purification of water-supplies; the best distribution of forests, woodlands, and other growth, and of cultivated and irrigated areas in their relation to river flow; the protection of forested and woodland areas from destruction by fire or insects; the deforestation of denuded areas; the planting of forests and establishments of forest plantations; the preservation and planting of woodlands and any other growth and protective cover on watersheds; the increase and development of the porosity and absorbent qualities and storage capacity of the soil upon which rain or snow may fall; the making and furnishing of plans for floodwater storage and other works for irrigation and power for farms, towns and villages; the acquisition, subdivision and settlement in small, intensively cultivated farms of lands for water storage by irrigation; the building of of the irrigation systems for such lands, including reservoirs, dams, canals, ditches, and all necessary works; the protection of farms, villages, towns and municipalities from damage by freshets and overflow; and the impounding of flood waters in artificial lakes and storage reservoirs to prevent floods and overflows, erosion of river banks, and breaks in levees, and to regulate the flow of streams and reënforce such flow during drought and low-water periods, the ultimate object of all such work being to regulate and, so far as possible, standardize the flow of rivers and their tributaries and source streams, and in the accomplishment of that object to induce and secure the coöperation of States, municipalities, districts, counties, towns, and other local agencies and organizations.

The various other governmental bureaus or departments represented on the Board, are each to have their respective duties. Thus, the Smithsonian Institution is expected to collate all available information of interest to the work of the Board, and for that purpose is allotted $1,000,000 per year out of the $60,000,000. The Bureau of Plant Industry is to study the soil and production of crops through the use of water as a fertilizer, the relation of water to successful crop production, etc., and is to have $6,000,000 per year. The Forest Service is to have charge of all that part of the work bearing upon the relation of forests to the various problems under consideration, and is to have $10,000,000 per year, one-half of which is to go toward the purchase of forest reserves. The Geological Survey is to have charge of topographic and geological surveys of interest to the Board and Commission, for which service it is allotted $3,000,000 a year. The Reclamation Service is to utilize the experience obtained in the construction of irrigation work, as information for the Board, and is to coöperate with works of that sort undertaken by the Board, particularly the reclamation of land by irrigation, drainage, or both, for the building of irrigation and drainage systems to aid in the regulation or equalization of the flow of rivers. For this pupose, the Reclamation Service is allowed $20,000,000. The Corps of Engineers of the United States Army is to present to the Board or propose plans for works to be built under the Act, and, under the direction of the Waterways Commission, is to supervise the construction of such works, directed toward the control and regulation of the flow of rivers and not toward the increasing of their navigability, it being provided that the Corps is still to be in charge of any work provided for by Congress under a separate River and Harbor Bill appropriation. For its part of the work, the corps of engineers is allotted annually $20,000,000. This uses up the entire $60,000,000 annual appropriation provided for in the Act.

In order to prevent as far as possible the methods now in force in public buildings and river and harbor work, the bill provides for equitable apportionment and contemporaneous execution of works among the several waterway systems in the United States; that is, the $60,000,000 annually is apportioned as follows, the divisions being more precisely specified in the bill than here.

To the Appalachian and Atlantic regions, $10,000,000; the drainage basin of the Ohio River, $10,000,000; the drainage basin of the Mississippi River above St. Louis, $5,000,000; the Mississippi River below St. Louis, $10,000,000; the Missouri River and tributaries, $10,000,000; the Colorado River, $5,000,000; the Sacremento and San Joaquin Valley, $5,000,000; the Columbia and Snake Rivers, $5,000,000.

The President is further authorized, in case there are insufficient funds to provide the $60,000,000 annual fund, to issue bonds bearing interest at the rate of 3% per annum, payable semi-annually, running for a period not exceeding 30 years.

It is evident from this resumé of the main points of the Newlands Bill, that it is a most ambitious project. If enacted, it would create a government engineering corps of great size and wide powers. It would appropriate for each of the next ten years, twice as much money as has been averaged in appropriations for the river and harbor work of this country in each of the past ten years. It would correlate all of the talent of the scientific bureaus of the Federal government, whose efforts, if rightly directed, would surely, before the end of the ten years allotted, have a most desirable influence upon river regulation troubles in this country.

In spite of this the bill should not be enacted. To be specific, the Newland's Bill is faulty because it seeks to appropriate so large a sum at a time when an economy-exploiting Congress is engaged in passing a revenue-reducing tariff bill; because it assumes that the subject of river regulation is inextricably woven up with reservoir control, forest increase and ground-water elevation, all subjects of the most controversial nature; because it lays upon the Federal government the administration of many constitutionally doubtful duties and, finally, because it is most uncertain in its clauses as regards the coöperation in costs and benefits between the Federal and the minor governments. Several pages of the bill are devoted to this matter of coöperation, but careful reading fails to show just how the Board or the Commission is to apportion the benefit or to divide the cost. With the government at Washington appropriating $60,000,000 each year, it can easily be seen how hard it will be to get any of the individual communities, counties or States voluntarily to put in very much money to help out the "widow's cruse of oil" at Washington. In addition the whole plan has a defect inherent in all Federal appropriations; that is, the opportunity for political skullduggery in the apportionment of the appropriation. While

an attempt has been made to avoid this trouble by a fair division of the "pork barrel," the subdivisions are still so large that the influence of prominent congressmen will be too great, and the bargaining back and forth between various parts of the country will become as indecent in the river regulation field as it has already in the case of public buildings and rivers and harbors.

The trouble with the bill is that, like Cæsar, it is too ambitious. Everyone who has made a study of the flood problem in the United States realizes that a comprehensive plan for the regulation of the rivers of the country is a future necessity. Sporadic efforts by local authorities can only result in temporary and local improvement, which surely fails when large areas are subjected to highly supernormal natural conditions, such as existed over the Mississippi and Ohio Valleys in the Spring of 1913. But to provide such a comprehensive plan is no easy task; certainly it cannot be done by mere legislative enactment, or by the casual writing of such a bill.

The engineering details of the control of one river, while not simple, are part of the technical equipment of the profession. When that one river is multiplied by a hundred, the technical difficulties increase, but not beyond the powers of the engineer, especially as he has the worthy example of several European countries to study. But when those hundred rivers are partly controlled by at least fifty different political units, each clamorous for benefits and each sensitive of its prerogatives and laws, the problem passes out of the engineer's hands into the lawyer's and economist's.

Any action by Congress in this important matter would better be taken as a result of a report after a careful investigation by a body of experts, engineering, legal and economic, and not be the snap appropriation of sixty millions of dollars to be devoted to the vaguest and at the same time the most general of activities.

LETTERS TO THE EDITOR

Concerning the Designer of the Poughkeepsie Bridge

Sir—The statement made in your issue of Aug. 7, p. 276, that the late Robert Escobar was the designer of the Poughkeepsie Bridge over the Hudson River is incorrect. The Poughkeepsie Bridge contract was awarded to the Union Bridge Co. on an estimate of cost and plans prepared by the members of the firm (Thomas C. Clarke, George S. Field, Edmund Hayes, C. S. Maurice, Charles Kellogg and Charles Macdonald were the founders and proprietors of the Union Bridge Co.) in New York City. This, of course, involved the proportioning of the piers and the general arrangement of the trusses as continuous spans.

The work of preparing the detail plans of the superstructure was carried out under the direction of Joseph Mayer (now Principal Assistant Engineer of the Quebec Bridge Commission, Montreal, Que.) in the Buffalo office of the firm. The late Robert Escobar had nothing whatever to do with that work, and Mr. Mayer is entitled to the credit of working out satisfactorily the part of the structure which was placed in his charge, but he is not, of course, responsible for the original conception.

Mr. Escobar was in my employ some years before the Union Bridge Co. was formed and remained under my particular direction until the absorption of the Union Bridge Co. by the American Bridge Co. in 1900. He had somewhat similar work to do in connection with the Hawkesbury Bridge in Australia that Mr. Mayer had on the Poughkeepsie Bridge, but Mr. Escobar's work was confined entirely to the mathematical questions involved. He was a brilliant mathematician, and in the early days of bridge designing, when the determination of stresses was not so generally understood as it is now, he was an admitted authority. He was continually evolving new and subtle methods of arriving at results; and I never knew him to make a mistake. He should be remembered as an eminently successful scientific investigator of the principles involved in bridge construction.

CHARLES MACDONALD.

Aug. 18, 1913.

[We are very glad to give space to Mr. Macdonald's letter correcting the errors of our previous article and doing justice to Mr. Mayer, whose connection with the Poughkeepsie Bridge design had temporarily escaped our memory. The statements which Mr. Macdonald corrects were made on what we believed to be first-class authority, and we regret that in making them we should have been unconsciously unjust to another engineer. Let us take this opportunity to impress upon our readers their obligations to the profession, and to the memory of their deceased co-workers, in placing on record some authentic record of those engineers who have helped to make the history of engineering.—ED.]

NOTES AND QUERIES

The size of steam pump used for the hydraulic sluicing jets on the dipper dredge described in our issue of July 31 (p. 197) was given as "9¼x10x12 in." This should be 9x14x 10x12 in.

In the table at the top of p. 283, "Engineering News," Aug. 14, 1913, in D. A. Watt's article on the "Operation of the Mohawk River Bridge Dams," the note on "Foundations" of Dam No. 4, at Scotia, should have read as follows:

Piers and cutoff wall on hard pan; all other portions in sand and gravel without piles, except that piles are used under the guide walls.

⁂

The Street-Lighting Contracts of the city of Philadelphia for the coming year will be arranged on a basis of payment according to the illumination rather than the lamps used or energy or fuel furnished. As noted in "Engineering News," June 5, 1913, p. 1197, under the direction of Hollis Godfrey, Chief of the Gas Bureau, the present administration equipped a portable (automobile) photometric laboratory which was used around the city to determine the degree of fulfillment of the old contract specifications.

Typhoid Fever in Large American Cities

By George A. Johnson*

The decrease in typhoid fever in large American cities has been very marked in recent years, and is largely due to the splendid advances which have been made in the general betterment of public water-supplies. Doubtless much credit must be given to improvement of sanitary conditions in general, and doubtless also a better understanding of the disease from the standpoint of medical treatment is responsible for the considerably lower mortality.

Recent correspondence with the health officials in the larger American cities has furnished fairly complete typhoid-fever statistics through the year 1912. The data thus far received from cities having in 1910 populations of over 100,000 are given in detail in Table I, and for convenience in discussing the effect of purification of the water-supply on the typhoid death rate, these data are also brought together in other places in such tabular or diagrammatic form that the effect of all forms and of each individual form of water treatment may be seen at a glance.

In the first column of Table II are included statistics from cities which, up to the close of the year 1912, had no form of artificial purification of their water-supplies. In the next column are contained the statistics from all cities using in 1912 some form of artificial purification, whether clarification, sterilization, or filtration through slow or rapid sand filters. Statistics from cities sterilizing their water-supply are given in the next column. The next two columns contain statistics from cities employing, in 1912, filters of the slow sand and rapid sand types, respectively. The last two columns include three selected cities, mentioned further on. The figures given in Table II are plotted in the accompanying diagram.

The inference is not to be drawn from the statistics presented in this table that the data are of value as showing precisely the effect of any particular method of purification employed, but rather the general effect of water purification. During the period covered by these figures many purification plants were installed, but not necessarily during the same year.

The year 1910 was a bad one for typhoid fever. There appears to have been a "wave" of this disease which was widespread in its effect, nearly one-half of the large cities having an abnormally high amount of typhoid fever during that year. In a number of cities sharp epidemics of this disease broke out, as at Kansas City, Milwaukee, Minneapolis and Omaha, and the installation of sterilization plants for the water-supplies of these cities followed promptly. This "typhoid wave" phenomenon is responsible for the high typhoid fever death rate in 1910, and particularly among the cities which are now using water sterilization plants.

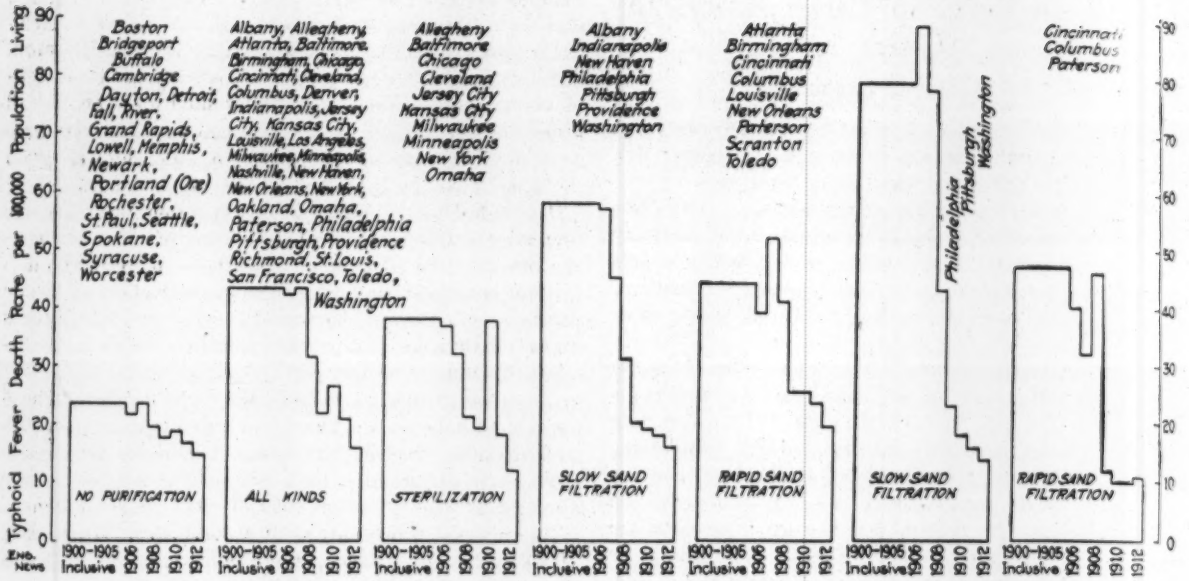
A number of large filtration plants have gone into service during the past five years, and their salutary effect on the typhoid-fever death rate is clearly shown. More definite information on this point is contained in the last

*Consulting Engineer, 150 Nassau St., New York City.

TABLE I. TYPHOID FEVER DEATH RATE PER 100,000 POPULATION LIVING IN CITIES OF OVER 100,000 POPULATION

City	Population in 1910	Average for the years 1900-1905, incl.	1906	1907	1908	1909	1910	1911	1912	Source of water-supply	Purification of water-supply†
Allegheny, Pa.*	150,000	107	136	97	40	28	42	39	9	Allegheny River	Sterilization
Albany, N. Y.	100,253	25	20	20	11	19	15	18	18	Hudson River	Slow sand filtration
Atlanta, Ga.	154,839	65	50	64	47	44	43	56	35	Chattahooche River	Rapid sand filtration
Baltimore, Md.	558,845	36	34	41	31	23	42	26	22	Gunpowder Cr. and Jones Falls	Sterilization
Birmingham, Ala.	132,685	50	39	71	64	44	51	44	38	Cahaba River	Rapid sand filtration
Boston, Mass.	680,585	23	22	10	26	14	11	8	9	Impounded streams	None
Bridgeport, Conn.	102,054	15	10	13	13	13	9	4	8	Poland River impounded	None
Buffalo, N. Y.	423,715	29	24	29	21	23	20	25	11	Lake Erie	None
Cambridge, Mass.	104,839	18	18	10	10	9	12	4	4	Impounded streams	None
Chicago, Ill.	2,185,283	27	18	18	15	12	14	10	7	Lake Michigan	One-fourth sterilized
Cincinnati, O.	363,591	54	71	46	19	13	6	11	7	Ohio River	Rapid sand filtration
Cleveland, O.	560,663	51	20	19	13	12	19	14	6	Lake Erie	Sterilization
Columbus, O.	181,511	61	45	38	110	17	17	14	19	Scioto River	Rapid sand filtration
Dayton, O.	116,577	29	28	38	16	24	18	18	17	Driven wells	None
Denver, Colo.	213,381	37	68	67	58	24	28	17	13	Platte River and Marston Lake	Infiltration galleries, slow and rapid sand filters
Detroit, Mich.	465,766	17	22	28	22	19	20	16	19	Lake St. Clair	None
Fall River, Mass.	119,295	19	8	18	13	14	13	15	16	No. Wattupa Pond	None
Grand Rapids, Mich.	112,571	34	39	30	30	17	26	25	31	Grand River	Rapid sand filtration
Indianapolis, Ind.	233,650	76	39	29	26	22	30	26	17	White River and wells	Slow sand filtration
Jersey City, N. J.	267,779	19	20	14	10	8	12	8	8	Rockaway River impounded	Sterilization
Kansas City, Mo.	248,381	48	38	40	35	23	43	24	12	Missouri River	Sterilization
Los Angeles, Cal.	319,198	35	18	23	19	18	12	13	12	Los Angeles River and wells	Infiltration galleries
Louisville, Ky.	223,928	55	63	79	49	43	31	25	19	Ohio River	Rapid sand filtration
Lowell, Mass.	106,294	19	7	9	24	11	21	6	9	Driven wells	None
Memphis, Tenn.	131,105	37	39	35	33	41	28	61	56	Wells	None
Milwaukee, Wis.	373,859	19	31	26	17	21	46	19	23	Lake Michigan	Sterilization
Minneapolis, Minn.	301,408	38	33	26	18	20	58	11	11	Mississippi River	Sterilization
Nashville, Tenn.	110,364	54	66	85	62	53	48	51	29	Cumberland River	Coagulation-sedimentation
Newark, N. J.	347,469	17	18	24	12	11	13	10	7	Pequannock River impounded	None
New Haven, Conn.	133,605	44	54	30	34	20	18	23	24	Lakes	Slow sand filtration (Lake Whiting only)
New Orleans, La.	339,075	40	30	56	31	25	32	30	13	Mississippi River	Rapid sand filtration
New York, N. Y.	4,766,883	19	15	17	12	12	12	11	10	Croton River impounded	Sterilization
Oakland, Cal.	150,174	19	26	28	18	7	13	13	19	Impounded	Rapid sand filtration
Omaha, Neb.	124,096	20	28	24	22	31	89	17	12	Missouri River	Sedimentation and sterilization
Paterson, N. J.	125,600	25	4	11	10	5	7	5	7	Passaic River	Rapid sand filtration
Philadelphia, Pa.	1,549,008	47	74	60	36	22	17	14	12	Delaware and Schuylkill Rivers	Slow sand filtration
Pittsburgh, Pa.*	410,000	132	141	135	53	13	12	10	6	Allegheny River	Slow sand filtration
Portland, Ore.	207,214	..	25	33	24	20	23	18	17	Bull Run River	None
Providence R. I.	224,326	20	19	8	16	12	17	14	10	Pawtucket River	Slow sand filtration
Richmond Va.	127,628	66	44	41	50	24	22	17	15	James River	Coagulation-sedimentation
Rochester N. Y.	218,149	15	17	16	12	9	14	11	12	Hemlock Lake	None
St. Louis, Mo.	687,029	33	18	16	15	15	13	10	13	Mississippi River	Coagulation-sedimentation
St. Paul, Minn.	214,744	14	21	17	12	20	20	10	9	Lake and artesian wells	None
San Francisco, Calif.	416,912	20	..	57	27	17	15	16	14	Impounded	Filtration—aeration
Scranton, Pa.	129,867	18	11	76	11	11	14	12	9	Impounded	Rapid sand filtration
Seattle, Wash.	237,194	..	20	48	22	23	14	10	8	Cedar Lake and River	None
Spokane, Wash.	104,402	..	47	38	40	32	32	37	17	Dug wells	None
Syracuse, N. Y.	137,249	14	10	16	15	12	28	15	16	Skaneateles Lake	None
Toledo, O.	168,497	36	45	36	40	31	37	22	31	Maumee River	Rapid sand filtration
Washington, D. C.	331,069	59	52	36	39	33	25	22	23	Potomac River	Slow sand filtration
Worcester, Mass.	145,986	17	12	14	10	8	16	7	3	Brooks and pond	None

*Allegheny and Pittsburgh are now combined in Greater Pittsburgh. †The rapid sand filtration plant at Atlanta, Ga., is of the pressure type; all the other rapid filters are of the gravity type.


Typhoid Fever Death Rate per 100,000 Population Living
Boston Bridgeport Buffalo Cambridge Dayton, Detroit, Fall River, Grand Rapids, Lowell, Memphis, Newark, Portland (Ore) Rochester, St Paul, Seattle, Spokane, Syracuse, Worcester
NO PURIFICATION
Albany, Allegheny, Atlanta, Baltimore, Birmingham, Chicago, Cincinnati, Cleveland, Columbus, Denver, Indianapolis, Jersey City, Kansas City, Louisville, Los Angeles, Milwaukee, Minneapolis, Nashville, New Haven, New Orleans, New York, Oakland, Omaha, Paterson, Philadelphia Pittsburgh, Providence Richmond, St. Louis, San Francisco, Toledo, Washington
ALL KINDS
Allegheny Baltimore Chicago Cleveland Jersey City Kansas City Milwaukee Minneapolis New York Omaha
STERILIZATION
Albany Indianapolis New Haven Philadelphia Pittsburgh Providence Washington
SLOW SAND FILTRATION
Atlanta Birmingham Cincinnati Columbus Louisville New Orleans Paterson Scranton Toledo
RAPID SAND FILTRATION
Philadelphia Pittsburgh Washington
SLOW SAND FILTRATION
Cincinnati Columbus Paterson
RAPID SAND FILTRATION
1900-1905 Inclusive 1906 1908 1910 1912
END. NEWS

TYPHOID-FEVER DEATH RATES IN LARGE AMERICAN CITIES, 1900 TO 1912, INCLUSIVE

two columns of Table II, wherein are given the results from three representative, modern slow sand filters, and the same number of rapid sand filters of modern design.

It was only a few years ago that a typhoid-fever death rate of 20, or less, per 100,000 population living, was considered a fairly reliable indication that the public water-supply was of satisfactory hygienic quality. Nineteen (37%) out of a total of 51 American cities of over 100,000 inhabitants had such a typhoid death rate in 1906, among a total population of 10,887,000; and 9 of the 19 had untreated water-supplies. In 1912 the average typhoid-fever death rate in these 19 cities was 10.

In 1912, in the 51 cities under discussion, and including a total population in 1910 of 20,337,000, the average typhoid-fever death rate was 15 per 100,000 population. In the same cities in 1906 it was 35. It must be remembered that this difference means about 4000 less lives lost from typhoid fever, and some 56,000 less people suffering annually from the disease in these cities.

Let us for the purpose of discussion retabulate the statistics for the 9-year period, 1900-1908, inclusive, and the 4-year period, 1909-1912, inclusive (see Table III).

TABLE II. TYPHOID FEVER DEATH RATE PER 100,000 IN AMERICAN CITIES HAVING POPULATIONS IN 1910 OF 100,000 OR MORE, SHOWING COMPARATIVE STATISTICS IN CITIES WHERE THE WATER-SUPPLY IS ARTIFICIALLY TREATED IN SOME MANNER, AND IN CITIES WHERE THERE IS NO TREATMENT.

Present method of artificial purification	None	All kinds	Sterilization	Slow sand filtration	Rapid sand filtration	Slow sand filtration	Rapid sand filtration
Number of cities....	13	30	10	7	8	3	3
Total population—1910	3,965,000	16,207,000	9,526,000	2,992,000	1,821,000	2,300,000	671,000
Average for 1900–5. Six inclusive years.	24	44	38	58	45	79	47
1906.....	22	43	37	57	40	89	40
1907.....	24	44	32	45	53	78	32
1908.....	20	32	21	31	42	43	46
1909.....	18	22	19	20	26	23	12
1910.....	19	27	38	19	26	18	10
1911.....	17	20	18	18	24	16	10
1912.....	15	16	12	16	20	14	11
Average for 1900–8. Nine inclusive yrs.	23	43	35	53	45	76	44
Average for 1909–12. Four inclusive yrs.	17	21	22	18	24	18	11
	Boston, Bridgeport, Buffalo, Cambridge, Dayton, Detroit, Fall River, Grand Rapids, Lowell, Memphis, Newark, Portland (Ore.), Rochester, St. Paul, Seattle, Spokane, Syracuse, Worcester.	Albany, Allegheny Atlanta, Baltimore, Birmingham Chicago, Cincinnati, Cleveland, Columbus, Denver, Indianapolis, Jersey City, Kansas City, Louisville, Los Angeles, Milwaukee, Minneapolis, Nashville, New Haven, New Orleans, New York, Oakland, Omaha, Paterson, Philadelphia, Pittsburgh, Providence, Richmond, St. Louis, San Francisco, Scranton, Toledo, Washington.	Allegheny, Baltimore, Chicago, Cleveland, Jersey City, Kansas City, Milwaukee, Minneapolis, New York, Omaha.	Albany, Indianapolis, New Haven, Philadelphia, Pittsburgh, Providence, Washington.	Atlanta, Birmingham, Cincinnati, Columbus, Louisville, New Orleans Paterson, Scranton, Toledo.	Philadelphia, Pittsburgh, Washington.	Cincinnati, Columbus, Paterson.

Note: Following cities having populations in 1910 of over 100,000 are not included in the above table, owing to the incomplete data: Dayton, Memphis, Oakland, Portland, San Francisco, Scranton, Seattle, Spokane.

TABLE III. COMPARISON OF TYPHOID-FEVER DEATH RATES IN CITIES WHERE THE WATER-SUPPLY IS UNTREATED, AND IN CITIES WHERE IT IS IN SOME MANNER ARTIFICIALLY PURIFIED

Number of cities		18	33
Present method of artificial purification		None	All kinds
Total population in 1910		3,965,000	16,207,000
Typhoid fever Death rate per 100,000 population living	Average for 9-year period, 1900–1908, inclusive	23	43
	Average for 4-year period, 1909–1912, inclusive	17	21
	Reduction	6	22

From the above we learn that in the 18 cities with untreated water-supplies the reduction in the typhoid-fever death rate was from 23 to 17 in the periods 1900-1908 and 1909-1912, respectively. Six less lives annually were sacrificed to typhoid fever for every 100,000 people living. In a sense, at least so far as the water-supply is a factor, this may be considered as the "natural" reduction in deaths from this disease due to causes other than purer water. During the same periods, in the cities where the water-supply was in some manner artificially purified, either by coagulation and sedimentation, sterilization or filtration, the reduction, measured similarly, represented 22 less lives lost annually from typhoid fever in every 100,000 people living. The difference in reduction, the chief cause of which must be credited to improvements in the quality of the water-supplies of the cities having treated water, was 16 typhoid fever deaths annually per 100,000 population.

It is virtually impossible to measure the value of a human life, but it is generally taken as $5000. Therefore, in the 33 cities with a total population in 1910 of 16,207,000, and with a net saving of 16 lives for each 100,000 people, about 2600 typhoid-fever deaths were prevented, which at an average value of $5000 represents an annual money saving of $13,000,000. Furthermore, assuming that the present-day mortality in this disease is about 7%, then some 14 people would suffer from typhoid fever to result in one death. Hence, to cause 2600 deaths some 39,000 people would have suffered from the disease. If we assume that lost time and medical attendance will average a cost of $150 for each case, we have the sum of $6,000,000 to add to the value of the lives saved, making a total saving of $19,000,000 annually, due to the reduction of typhoid fever alone.

It is fair to say that the total cost of water filtration, including the interest on the investment for filtration works, cost of operation, supplies, repairs and replacements, will amount on an average to about $10 per million gallons of filtered water. In the 33 cities under discussion, allowing a consumption of 150 gal. per capita daily, there would be required 887,315 million gallons annually. At $10 per million gallons this represents a total cost of $8 873,150. It has been shown above that pure water was unquestionably chiefly responsible for a saving in vital capital of $19,000,000 annually, or over twice the amount of money required to furnish pure filtered water for all of the 33 cities.

Only 17 of these cities had filter plants in 1912, the other half getting along with the partial purification afforded by sedimentation and sterilization. Grand Rapids finished its rapid sand filter plant late in 1912; a plant of the same type has just been finished at Minneapolis; filtration for Cleveland is being actively discussed; Allegheny will be supplied with filtered water from the Aspinwall plant (Pittsburgh) within a year; and contracts have been let for the construction of rapid sand filter plants at Baltimore and St. Louis. Can there be any question of the wisdom of such investments?

By far the most spectacular results in the line of typhoid-fever reduction have been recorded at Pittsburgh, Penn. The records show that for years that city stood at the head with respect to the widespread prevalence of typhoid fever, the annual death rate from this disease easily being four times the average for all other large American cities. A modern slow sand filter plant was completed in 1908, during which year a part of the city was supplied with filtered water; and in 1909 practically all of peninsular Pittsburgh. The Allegheny River, from which the city takes its raw supply, is badly polluted with sewage and industrial wastes, yet by careful treatment the water-supply of Pittsburgh now ranks among the safest in the world. Following the same line of argument as set forth above, and reducing the benefits of filtration at Pittsburgh to dollars and cents, it is seen that the purification of the water-supply is preventing some 7000 typhoid cases and saving some 500 lives each year, representing a money value of $3,500,000. Two years' saving in vital capital effected in this manner is equal to the total cost of building the purification works, which are good for a generation, at least.

Water-Works Organizations and Operating Methods; St. Paul, Minn., and San Diego, Calif.

The accompanying diagrams illustrate the municipal water-works organization schemes of two typical American cities; St. Paul, Minn., with a population of about 222,700, and San Diego, Calif., with about 40,000 people. In St. Paul some 59% of the services are metered, and the receipts from metered water are nearly 73% of the total. San Diego enjoys the distinction of being one of the few American cities which has all its water services metered. Both cities have live, uptodate water departments, which are constantly striving for efficient service and reasonable per capita consumption.

St. Paul, Minn.

Early in the year 1912 the Board of Water Commissioners of St. Paul employed the Bureau of Municipal Research, of New York City, to make a preliminary survey of the business and accounting methods of the water department. The recommendations of the Bureau were acted upon, and as a result definite plans of reorganization were made and adopted. The organization chart, Fig. 1, and a brief description are taken from the recently issued Annual Report of the Water Department.

There are three main functional divisions or bureaus: (1) Bureau of water-supply; (2) Bureau of water revenue; (3) Bureau of accounts. The bureau of water-supply is administered by the general superintendent, who in addition acts in a consulting capacity to the other bureaus. The bureau of water revenue is administered by the water registrar, who has control over the five subdivisions: (*a*) applications and permits; (*b*) inspection; (*c*) consumers' accounts; (*d*) collection; (*e*) information and complaints. The bureau of accounts, which also includes the purchase and inspection of supplies, is

administered by the secretary and accountant; through this bureau an accounting control of all revenue is maintained, and through it all reports to or directions from the Board of Water Commissioners are transmitted.

The administrative control of the bureau of water-supply covers six subdivisions: (*a*) engineering; (*b*) collection and storage; (*c*) pumping; (*d*) distribution; (*e*) meters; (*f*) clerical work. Each subdivision has one or more heads who receive orders from and report directly to the general superintendent of water-supply. The activities of the whole bureau have been coördinated so as to give the very best results. Accurate records are kept showing the cost of each separate piece of work.

In order to carry on the work of the various divisions to the best advantage the office arrangements were revised. The rules and regulations of the water department were revised wherever necessary. The accounting system was revised so as to give one division complete control over all revenues. Clerical work has been materially reduced by the use of addressing and adding machines. An annual budget is now prepared appropriating definite sums of money for accomplishing the work estimated for by the different bureaus and subdivisions.

The meter rate for the present year (1913), .6c. per 100 cu.ft., is based on a careful analysis of the revenues of 1912, and it is estimated will furnish sufficient revenue to pay the operating expenses of the water department, make the necessary repairs and replacements to the present equipment, pay the interest on the bonded indebtedness, provide an additional amount for a sinking fund as provided by law for the redemption of outstanding bonds, and leave a safe and adequate surplus; provided funds for the acquisition of new property, equipment, extension and improvement of present works are obtained from the sale of water-department bonds. The city further pays the water department a rental of $14 per hydrant per year.

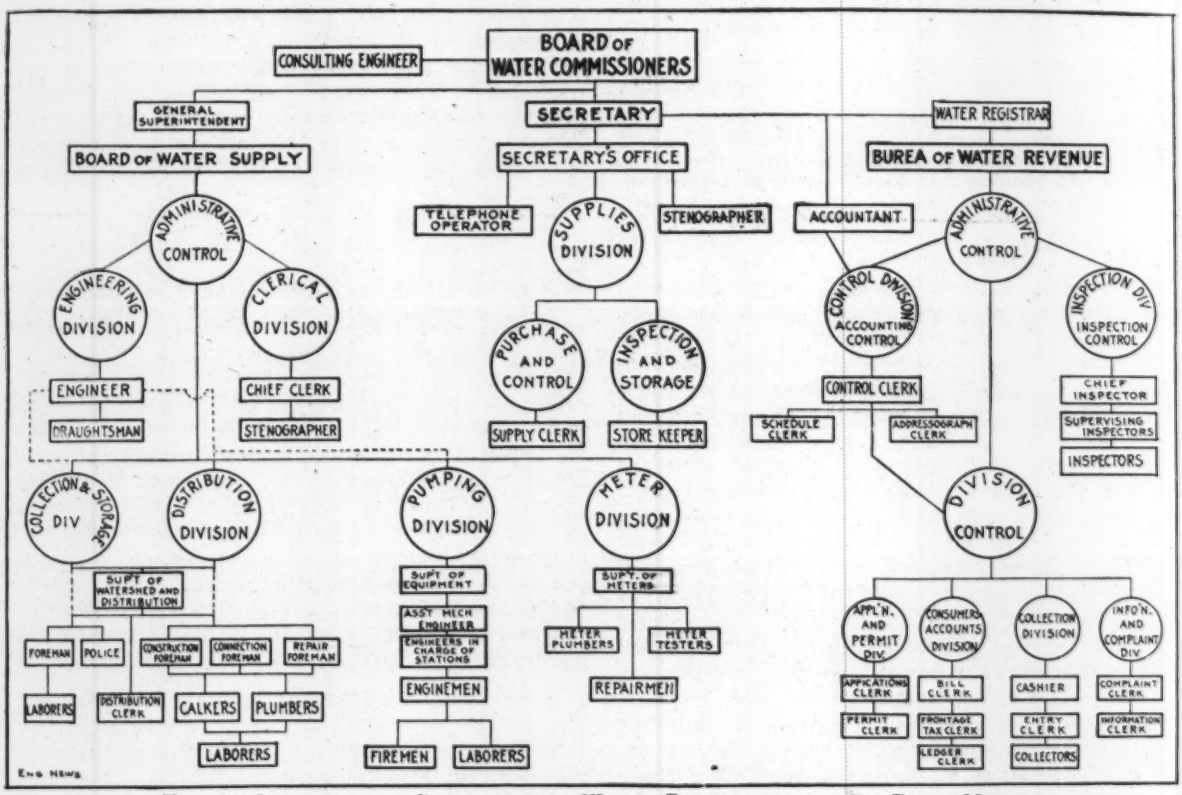


FIG. 1. ORGANIZATION CHART OF THE WATER DEPARTMENT OF ST. PAUL, MINN.

SAN DIEGO, CALIF.

The organization of the water department is shown very clearly by the chart, Fig. 2, and no explanation is necessary. We are indebted to H. A. Whitney, Assistant Superintendent, for the following information in regard to work accomplished by the water department.

The smallest service connection is a ¾-in. iron pipe, dipped in a specially prepared asphaltum bath, which has been found preferable to galvanized-iron pipe in the soil of San Diego. The charge to the consumer for a service connection consisting of a corporation cock with a lead connection and line of ¾-in. pipe to a ⅝x¾-in. meter, set in a wooden meter box with a specially designed cast-iron cover, is $18. A record of each meter installation is kept on a card, shown in Fig. 3, which gives the date, reading and percentage of error from full flow down to $\frac{1}{32}$ flow. On the reverse of the card is the date installed, address of the consumer or property owner, order number, date taken out, record of repairs and cost of same.

Every meter is tested before installation. The specifications call for a meter that shall register within 2% of accuracy on a $\frac{1}{32}$ flow. In testing the small ⅝x¾-in. sizes a series of six are connected together and used in one test. Ten cubic feet of water are run through the meters into a tank, the water is weighed and the amount

compared with the meter readings. The error thus found is recorded on the card, Fig. 3.

If a complaint comes that a meter is dead or registers wrong a meter-repair tag is sent by the bureau of accountability to the foreman of operation. This tag is a printed blank form on heavy cardboard 6x4 in., and on it is recorded on one side the reading, meter number, make, removed ———— from ———— by reason of, time required on job; new meter number, style and size, order number. On the reverse side are blanks for the detail statement of the nature of the repairs, cost of the same and materials.

If it is found that the meter registers correctly and it was investigated through a complaint of the property

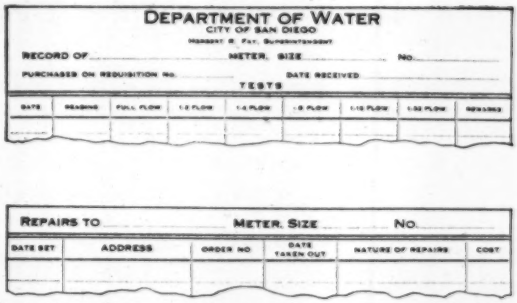


FIG. 3. TWO SIDES OF METER-RECORD FILING CARD, SAN DIEGO WATER DEPARTMENT

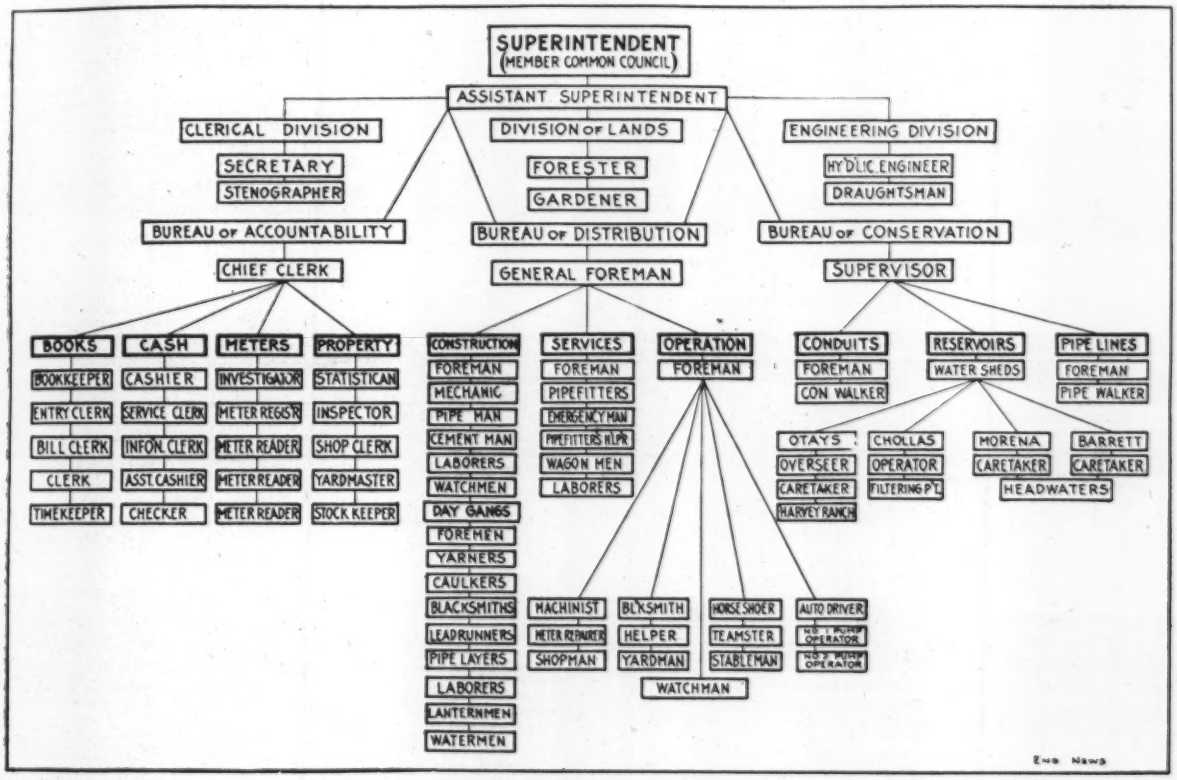


FIG. 2. ORGANIZATION CHART OF THE WATER DEPARTMENT OF SAN DIEGO, CALIF.

owner the cost of the removal and investigation of the meter is assessed against him. If the meter is actually not registering correctly the cost of the investigation and repairs are paid by the water department. A small automobile is used to carry all meters tagged for repairs to headquarters.

The meter rate fixed by city ordinance is 8c. per 100 cu.ft. The minimum monthly rate for all water furnished varies from $1 for a ⅝-in. meter to $5 for a 6-in. meter and larger sizes. All water furnished the various departments of the city are charged to their respective accounts and paid for from their special funds into the "water fund." With the consent and approval of the Common Council special rates of less than 8c. per 100 cu.ft. may be granted to public works, charitable institutions, and others in special instances; but in every instance where a special rate is granted the balance between it and the regular rate must be paid the water department out of the "general fund."

For furnishing, setting, maintenance, repairs and supervision of public fire hydrants a charge of $2.50 a month is made for each hydrant in service, this sum being paid monthly out of the "general fund" of the city. The city also pays out of the "general fund" $2.50 a month for each public drinking fountain, watering trough and standpipe. The other city services are metered just as private consumers.

A Reinforced-Concrete Pontoon Crane, 100 ft. long by 43 ft. wide, fitted with a motor and screw and capable of being navigated by its own power, has been constructed at Marseilles. It is to carry a crane 33 ft. high, with an overhanging or unsupported length of 33 ft., able to support a load of 30 tons. The pontoon has double sides, protecting it against the influx of water should the outer skin be damaged. —"The Times Engineering Supplement," London.

Drastic Provisions of the San Diego, Calif., Water-Works Ordinance

The City of San Diego, contemplating a $955,000-bond issue for installing steel pipe lines, building new concrete reservoirs, pumping stations, wells and distributing systems, and at present engaged in expending $340,000 in laying several miles of cast-iron mains, installing fittings and building three small reservoirs, has recently (Apr. 28) passed an ordinance covering the use of the city water-supply, which will be read with considerable interest by water-works engineers and managers because of its rather drastic provisions to prevent water waste.

San Diego, with a population of 40,000, is one of the few American cities to have 100% of its services metered. The meter rate is 8c. per 100 cu.ft.; and other city departments are treated exactly as ordinary consumers.

Some of the most interesting provisions of the new ordinance are reprinted below:

CONSTRUCTION WORK

For water furnished for construction work where meters are not installed or used, the rate to be charged shall be as follows:

(a) For mixing and wetting concrete used in street paving, $1.25 per 1000 sq.ft. of paving laid.

(b) For sidewalk and curbing, $1 per 1000 sq.ft. of concrete laid.

(c) For mixing concrete for any other construction work, not herein provided for, the rate shall be 5c. per bbl. of cement used.

(d) For settling earth and ditches, ½c. per cu.yd. of trench or excavation.

(e) For water supplied for street grading or any construction work not otherwise specified in this ordinance, when not used through a meter, the amount of water used and the charges for same shall be fixed by the Superintendent of the Department of Water.

(f) Contractors or any person desiring to use water in construction work, where connections must be made with city hydrants or standpipes, shall in each and every instance obtain a written permit from the Superintendent of the Department of Water before connecting with any such hydrant or standpipe, or using water therefrom, and such permit shall be exhibited upon the work for which issued.

(g) In each and every instance enumerated in subparagraphs a, b, c, d, e and f, the amount of the charge shall be estimated by the Superintendent of the Department of Water.

PAYMENT OF WATER BILLS

(1) For the payment of all water bills the city is divided into three districts, to be designated and known as Districts 1, 2, and 3, and all bills for water used shall be payable at the office of the Department of Water, at the City Hall.

(2) For water furnished in District No. 1, the rate shall be due on the first day of the month; in District No. 2, on the 11th day of the month; in District No. 3, on the 21st day of the month.

PENALTIES FOR DELINQUENTS

Should an account not be paid within 10 days after coming due, a penalty of 10% shall be added to the bill, and if the total amount then due is not paid within 10 days after the adding of the penalty, the said account and penalty shall be deemed delinquent and an additional penalty of 50c. shall be added, and the Superintendent of the Department of Water may cause the water to be shut off from the premises from which the account is delinquent, and the water shall not again be turned on until all arrearages and charges shall have been paid.

RATES LIEN AGAINST PROPERTY

In addition to any other remedy provided herein for the enforcement and collection of any water rate, charge or account, all rates provided for in this ordinance shall be charged against the property on which it is furnished, and against the owner thereof, and shall be a lien against the premises to which any water may be supplied, and a charge against the owner thereof and the occupant thereof using the water, and if for any cause any sums owing therefor become delinquent, the water shall be shut off, and in no case shall it be turned on to the same property until all such delinquencies shall have been paid in full, and such property owner and occupant shall be severally responsible to the City in an action waged by the City in any court of competent jurisdiction for the amount of all such rates as may be due and unpaid, together with all penalties provided herein and costs. No change of ownership or occupation shall affect the application of this section.

FORM OF CONTRACT

The application shall contain a contract, on the part of the applicant to pay the stipulated rates at the time and in the manner provided for in this ordinance, reserving to the City of San Diego the right to enforce and collect rates and penalties in the manner provided for in this ordinance, to change rates and to temporarily discontinue the service at any time without notice to the consumer, and further to provide that the Department of Water of the City of San Diego shall not be responsible for any damage by water or otherwise, resulting from defective plumbing, broken or faulty services, or water mains.

INSTALLATION OF SERVICE

The applicant having complied with all the requirements of this ordinance relating to application for meter and service, and having paid all charges incurred, the Department of Water shall cause the property described to be connected with the city water mains, in the following manner:

PROPERTY ON WATER MAINS

Where the property is located upon a street in which a water main of not less than 2 in. in diameter is laid, same shall be connected with the water system by a service pipe extending at right angles from the main to the curb line, a stop cock placed inside the line of the street curb, with a meter, meter box and cast-iron meter-box cover, provided, however, nothing in this paragraph shall interfere with the Department of Water installing a service pipe of sufficient size from the water main to the curb for the purpose of leading branches to the right and left in the parking to supply not more than three separate properties.

LIMIT TO EXTENSIONS

Where the property is located on a street in which there exists no water main in front of the premises applied for, the Superintendent of the Department of Water shall extend the nearest water main in the direction of said property, provided that in no instance shall an extension of any water main exceed 100 ft. for any one applicant who has in all other respects complied with the requirements of this ordinance, excepting that the Superintendent of the Department of Water may estimate the cost of any extension in excess of the 100 ft. allowed by this ordinance and upon payment by the applicant of the charges estimated, the same may be installed by the Department of Water, provided:

LIMIT OF SERVICES

(a) That only one service connection shall be allowed upon a ¾-in. pipe, having a pressure of less than 20 lb., two services upon a ¾-in. pipe with pressure between 20 and 60 lb., and three services upon a ¾-in. pipe with over 60 lb. pressure.

A 1-in. pipe may be allowed twice the number of service connections as a ¾-in. of the same pressure.

A 1½-in. pipe may serve twice the number as a 1-in., and a 2-in. pipe twice that as the 1½-in.

(b) No water pipes laid in the streets of this city of less than 2 in. in diameter shall be considered as mains, and the Superintendent of the Department of Water may at any time order same removed and a larger pipe installed.

COST OF SERVICE CONNECTIONS

The Department of Water shall make the following charges for the perpetual maintenance of all services, meters and meter boxes installed:

COST OF SERVICE CONNECTIONS

Size of Service	Meter and Box	Cost of Service	Service & Meter Paving	Service & Meter	Service, Meter & Paving
⅝-in.	$ 8	$10	$17	$18	$27	$35
¾-in.	14	10	17	24½	27	41
1-in.	20	12	20	32	32	52
1½-in. ...	37	20	21	57	41	78
2-in.	87	32	22	99	54	121
3-in.	137	100	24	237	124	261
4-in.	250	126	26	376	152	402
6-in.	500	166	30	666	196	696

These prices shall include the service connection complete, the meter set in place and an adequate meter box with a metal cover.

FIRE SERVICES

(a) Where services are used, or are to be used for sprinkler and fire service only, a proportional meter may be installed in place of the standard type meter; a reduction from the regular meter charges shall be made as follows:

(b) Upon a 2-in. service the reduction shall be $27.

Upon a 3-in. service the reduction shall be $77.

Upon a 4-in. service the reduction shall be $170.

Upon a 6-in. service the reduction shall be $400.

(c) In no instance shall any connections be made with any sprinkler or fire service without the written consent of the Superintendent of the Department of Water, and should it be found that any unauthorized connection has been made, or that water has been used from a sprinkler or fire service for any other purpose than extinguishing a fire, the water service shall be shut off and not again turned on until a sum equivalent to the monthly minimum for a similar sized regular installation is paid covering the entire period during which the service has been installed, and, furthermore, a standard meter of the equivalent size of the service pipe shall then be installed and paid for by the owner of the property affected.

WATER TO PASS THROUGH METER

All services, not otherwise provided for in this ordinance, shall be metered, and all city water used on any premises where a meter is installed shall pass through the meter.

No bypass or connection between the meter and the main shall be made, maintained or permitted, except as may be installed by permission of the Superintendent, Department of Water, for sprinkler or fire service.

REPAIRING METERS

All water meters installed by the Department of Water shall remain at all times the property of the City of San Diego, and shall be maintained and repaired when rendered unserviceable through fair wear and tear, and renewed by the Department of Water; provided, that where replacements, repairs, or adjustments of any meter are rendered necessary by the act, neglect or carelessness of the owner or occupant of any premises, any expense caused to the Department thereby shall be charged against and collected from the owner of the premises.

TESTING METERS

Any consumer shall have the right to demand that the meter through which water is being furnished be examined and tested by the Department of Water, for the purpose of ascertaining whether or not it is registering correctly the amount of water which is being delivered through it by the Department of Water to such consumer; provided, that when any consumer desires to have said meter so examined and tested, such consumer shall make application therefor in writing to the Department of Water, and shall deposit with the application the amount charged for the various sizes, to-wit:

(a) For testing a ⅝-in. meter, $1.
For testing a ¾-in. meter, $1.25.
For testing a 1-in. meter, $1.50.
For testing a 1½-in. meter, $1.75.
For testing a 2-in. meter, $2.
For testing a 3-in. meter, $3.
For testing a 4-in. meter, $4.
For testing a 6-in. or larger meter, $5.

(b) Upon the application being made it shall be the duty of the Department of Water to cause the meter to be examined and tested for the purpose of ascertaining whether or not it is registering correctly the water being delivered through it.

If on such an examination and test the meter shall be found to register over 3% more water that actually passes through it, another meter will be substituted therefor, and the fee charged in the application for a test shall be repaid to the person making the application.

METER NOT REGISTERING

Whenever a meter gets out of order, and fails to register correctly, the consumer shall be charged with an average daily consumption as shown by the meter when in good working order and registering correctly.

CONDITIONS OF SERVICE

For the convenience of the consumer and as a measure of safety to the service the following regulations are enacted and made a part of this ordinance.

WATER WASTE

It shall be unlawful for any person or persons to willfully or neglectfully waste water in any manner whatsoever, and any person having knowledge of any condition whereby water is being wasted shall immediately notify the Department of Water of such fact.

APPROVED SERVICE CONNECTIONS

The refusal or neglect of the owner or occupant to equip and maintain the premises with service pipe connections, utilities or fixtures of approved character and quality to prevent waste of water, shall be sufficient grounds for the refusal of the Department of Water to connect the premises with the city water supply, or to continue such connection after having given three days' written notice of intention to shut off water pending repairs because of water waste.

EMERGENCY SHUT-OFFS

Consumers shall install an improved shut-off inside the property line, at a location accessible in case of emergency.

CHECK VALVES

If in the judgment of the Superintendent, Department of Water, the placing of an approved check valve on the property side of a water meter of any consumer is considered necessary for the safety of the water system, such approved check valve shall be immediately installed, at the expense of the consumer, after due notice in writing shall have been given to the consumer by the Superintendent, Department of Water.

BOILER CONNECTIONS

It shall be unlawful for any person or persons to draw water from any city pipes direct into any stationary steam boiler, within the corporate limits of the City of San Diego.

SUPPLY TANKS

Where city water is used to supply a steam-engine boiler, its owner must provide a tank of sufficient capacity to afford a supply for at least 12 hours, into which the service pipe will be discharged.

IN CASE OF FIRE

Consumers are required to shut off lawn sprinklers or any steady flow of water they may be using whenever a fire occurs in their section of the city, or a large fire occurs in the business section of the city

TURNING ON WATER

It shall be unlawful for any person other than the authorized agents or employees of the City of San Diego to turn on water after the same shall have been turned off as herein provided, without the consent of the Superintendent, Department of Water; also for any person, excepting as above mentioned, or the proprietor of any grading outfit, to take or use water from said system of water-works, without the written consent of the Superintendent, Department of Water, and all owners and occupants of property are hereby prohibited from furnishing water for such purposes until the said Superintendent, Department of Water, has given permission so to do.

SHUTTING OFF WATER

Upon the application of the owner or occupant of a building or premises, to have the water shut off on the supply side of a meter, the Department will shut the water off and at the same time record the reading of the meter, for which service a charge of 50c. will be made.

UNLAWFUL USE OF WATER

It shall be unlawful for any consumer to supply water to any person, company or corporation other than the occupant or occupants of the premises of said consumer, or to knowingly permit leaks or wastage of water or for any person, company or corporation to tap, open or connect with any water main or pipe which forms any part of the system of the water-works of the City of San Diego, without first obtaining written permission so to do from the Superintendent, Department of Water.

SEWERS

No sewer pipe or sewer ditch shall be placed nearer than 2 ft. to any water-service pipe connected with this system of water-works.

FIRE HYDRANTS

(1) Fire hydrants are to be opened and used only by the Water and Fire Departments or such persons as may be specially authorized by the Superintendent, Department of Water.

(2) To insure the safety of fire hydrants for fire protection, any person or persons authorized to open fire hydrants will be required to use only an approved spanner wrench, and to replace the caps on the outlets when the same are not in use; failure to do so will be sufficient cause to prohibit further use of the hydrants and the refusal to grant subsequent permits for the use of fire hydrants.

(3) It shall be unlawful for any person to conduct or carry away any water from any fire hydrant without a written permit from the Superintendent, Department of Water, of said city.

(4) It shall be unlawful for any person or persons to place upon or about any fire hydrant, gate valve, manhole, stop cock, meter or meter box connected with the water pipes of the said system of water-works any object, material, debris or structure of any kind so as to prevent free access to the same at all times.

PRIVATE WATER SYSTEMS

(1) All water pipes laid in the streets, alleys or public thoroughfares within the corporate limits of the city of San Diego, shall become the property of the City of San Diego.

(2) All water systems laid within the corporate limits of the City of San Diego, and connecting with or supplied water from the water mains of said city, shall be constructed under the supervision and in accordance with plans and specifications approved by the Superintendent, Department of Water.

(3) It shall be unlawful for any person, persons or corporation to sell water for domestic purposes within the corporate limits of the City of San Diego, without having first received permission so to do from the Common Council by application approved by the Superintendent, Department of Water.

VIOLATIONS—CONVICTIONS—PUNISHMENT

Any person, persons, or corporation violating any provision of this ordinance shall be deemed guilty of a misdemeanor, and upon conviction thereof shall be punished by a fine in a sum not exceeding $500 or by imprisonment in the City Jail for a period not exceeding 50 days, or by both such fine and imprisonment.

INSTALLATION OF WATER MAINS AND FIRE HYDRANTS IN NEW TRACTS AND SUB-DIVISIONS OF LAND

(1) Whenever any tract or sub-division of land within the corporate limits of the City of San Diego, Calif., shall be laid out in lots for the purpose of sale, the owner or owners thereof, in addition to complying with all the requirements of the General Laws of the State of California, and other ordinances of the said City of San Diego, relating thereto, shall comply with the following additional requirements, to-wit:

(2) Whenever, in the opinion of the Common Council, it shall be deemed necessary or practicable, the owners of the land, of which the map or plat has been submitted, shall before the said map or plat is adopted, install water mains of sufficient size to insure adequate service to the number of lots specified in such proposed additions, sub-division or tract.

(3) Also, install the regulation fire hydrants, sufficient in number to secure proper fire protection in such proposed addition, sub-division or tract. Provided, however, that the Common Council may, in its discretion, accept in lieu of such finished work, a cash deposit or bond in such sum, as it deems sufficient as a guaranty that such water mains and fire hydrants will be installed and completed within such time as the Common Council may designate. Upon the approval of such bond or deposit, of such cash, the Common Council may accept and approve such map or plat, before the actual installation of such water mains and fire hydrants. All work of installing such water mains and fire hydrants shall be done to the satisfaction of the Common Council, and under the supervision of such inspectors as this Common Council may provide.

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The Distillation of Oil from Bituminous Shales—Some months ago, in our issue of Oct. 17, 1912, we reviewed the possibilities in the securing of large quantities of liquid fuel from the deposits of oil shale in the Mississippi Valley and elsewhere. It is of interest to note that notwithstanding the competition of cheap petroleum, extraction of oil from shale in Scotland is still carried on. In 1911, according to a review of the industry in the Engineering Supplement of the London "Times," 3,116,000 tons of shale were mined in Scotland and distilled for oil production. It is said that the average production of oil is about 25 gallons per ton of shale. The crude oil yields, on fractional distillation, over 3 per cent. of naphtha, about 25 per cent. of burning oils, 15 per cent. of gas oils and 15 per cent. of lubricating oils. The increased demand for oils for use in oil engines makes the future development of the oil shale industry very promising.

The Proposed Reforestation of the Cedar River Watershed, Seattle Water-Supply*

Few cities could have been more fortunate in the selection of a municipal water-supply than the city of Seattle, Wash. The source chosen some years ago by the city engineer was that supplied by the watershed of Cedar Lake, Cedar River and its tributaries, situated in the heart of the Cascade Mountains about 45 miles from the city. Here is a pure and practically unlimited supply of water, which apparently is to prove a source of a large future revenue as well.

Cedar Lake is 1547 ft. above sea level. Leaving the dam at the outlet of Cedar Lake, the Cedar River falls 600 ft. in a distance of 3½ miles, and at the foot of this descent the city of Seattle completed in 1909, a municipal hydro-electric power plant having a present normal capacity of 11,000 hp. and to be increased to an ultimate normal capacity of 20,000 hp.†

This hydro-electric station supplies current for the city street lighting and to many private consumers as well. Its profitableness as an investment of the public money

FIG. 1. MAP OF THE CEDAR RIVER WATERSHED, SEATTLE, WASH., WATER SUPPLY

has been amply proved; and it has been of much indirect profit to the city taxpayers in reducing the rates charged by private lighting and power companies.‡

Another perfectly feasible scheme to still further utilize the water-supply property is now proposed, which if carried out in detail is expected to yield the city an ultimate clear profit at the end of 60 to 80 years of at least $6,000,000 or $7,000,000. This scheme is the reforestation of the watershed lands, which have all the necessary qualities for regenerating a splendid forest. No better proof of their adaptability for forestry is necessary than the fact that a large part of the land is still covered with thousands of dollars worth of standing timber, and until very recently practically all the lands in the watershed were so covered. While various other city water-supply watersheds have been reforested as a method of utilizing otherwise useless lands and for protecting the purity of the water, most of this work has been on a small scale and few such schemes could ever have been entered into from so purely a commercial point of view and with so certain a prospect of a large financial return.

The Cedar River watershed is divided naturally into two main divisions—the upper watershed and the lower watershed (Fig. 1). The upper watershed extends from the site of the proposed new masonry dam, at a point on the Cedar River about two miles below the outlet of Cedar Lake, to the summit of the Cascade Mountains, covering an area of about 83 sq.mi. The lower watershed extends from the site of the proposed dam to Landsburg, at the intake of the water-works pipe lines to the city, and covers an area of about 54 sq.mi.

The upper watershed is practically the watershed of Cedar Lake. It is characterized by steep precipitous mountain slopes rising abruptly from the shores of Cedar Lake, and from the narrow valley of the Cedar River, and of the Rex River, to the summits of the flanking ridges on each side. In many places there are extensive areas covered by masses of rock, either in the form of cliffs or what is known as rock slides.

Near the summits of some of the mountains there are open parks and grassy meadows in which little timber is found; but aside from the barren spots the entire area is covered with a heavy growth of timber. This timber varies in character from stands of Douglas fir in the valley, where a single tree often reaches a height of 250 ft., with a diameter of 6 ft. at the base, to a scattering and scrubby growth of what is known as the upper slope type of timber upon the summits of the mountains. It is estimated that the present stand of commercially valuable timber on this upper watershed exceeds 1,000,000,000 ft., b.m.

The lower watershed, although bounded on the north, east and southeast by mountain ridges, is what may be characterized as rolling or bench land. The timber which originally covered the lands of this lower watershed was, generally speaking, of a larger growth and more valuable than that now standing upon the upper watershed; because the lower elevation of the lower watershed is more favorable to timber growth and affords a longer growing season. By reason of its proximity to the railroads and of its general accessibility, logging operations in the lower watershed were begun at an early period and a considerable portion of the area has now been denuded of timber and is ready for reforestation.

At the time the city purchased the lands of the watershed there was not sufficient money available to buy the standing timber belonging to the several logging companies already established there, or to buy the sawmills and equipment of the companies. Hence the operations of logging and cutting up timber have been carried on in the lower watersheds by private parties up to the present time and are still being carried on under such regulation that the danger of pollution of the water-supply has been reduced to a minimum. Within the upper watershed, however, no logging operations had been commenced previous to its acquisition by the city, and up to this time the city has been successful in preventing the establishment of logging camps and sawmills in this territory.

In order to do so it has been necessary in many instances for the city to purchase not only the land but also the timber standing thereon. In this manner the city has acquired the ownership of standing timber having an estimated value at the present time of nearly $700,000. The total cost of acquiring all the land and timber in the entire watershed up to this time, exclusive of the cost of surveys, appraisals and court costs, is something over $600,000, or nearly $100,000 less than the

*Based on a Report on the Reforestation of the Cedar River Watershed made by R. H. Ober, Superintendent of Buildings and H. C. Johnson, Forest Engineer, February, 1913.

†See "Engineering News," Dec. 14, 1911.

‡See "Engineering News," Aug. 29, 1912.

cost of the standing timber alone. Adding the cost surveys, appraisals, timber cruising and court costs in condemnation proceedings, etc., and it is found that the value of the standing timber now belonging to the city is approximately equal to the cost of acquiring the ownership of the watershed.

FIG. 2. VIEW OF CEDAR LAKE, SEATTLE'S MOUNTAIN WATER SUPPLY

The reforestation of the cut-over lands of the lower watershed does not require the continued residence of any large number of people, nor the establishment of a resident population with its attendant dangers, as would be the case if these lands were devoted to agriculture or other industrial uses to which they would otherwise be suited. Moreover, reforestation would result in a great benefit to the quality of the water as well as to conserve the supply to some extent by regulating the runoff of the stream.

The production of a growth of valuable timber on this land will result in the creation of an asset of enormous value to the country at large in place of a doubtful asset, or possibly a liability, if no systematic effort is made toward reforestation, and the area is allowed to grow up to weeds and a scattering growth of inferior timber, having but little value.

In order to determine the feasibility and advisability of adopting such a plan for the reforestation of this area, a careful study of the watershed has been carried on for four years by R. H. Ober, superintendent of buildings, and also during the last year by H. C. Johnson, forest engineer, who has made an exhaustive examination of the entire area of the lower watershed. The result of their work is the subject of an interesting report recently submitted to the municipal government of Seattle and is the basis of this article, the remainder of which, with some rearrangement and condensation, is virtually reprinted from the report:

The lands in the lower watershed logged off and burned over are all barren and non-restocking, and if left alone and the present conditions of annual fires allowed to prevail in the future, they must of necessity remain barren. The area of such lands is 24.6 sq.mi. All this land is most favorably situated with regard to the natural requirements of reforestation. Of this 1.2 sq.mi. is not taken into account as it comprises such areas as railway rights-of-way, that required for electric power transmission lines, and other areas which cannot be reforested. This leaves 23.4 sq.mi. in the lower watershed which could be reforested immediately.

The remaining 32.2 sq.mi. are still timbered, but two lumber companies are now cutting at the rate of 1000 acres a year, and when they are finished, with the addition of what other logging will have to be done on other holdings, there is sure to be approximately 24.3 sq.mi. more of logged off and burned over lands in the near future. On the other 8 sq.mi. now forested, the timber is so poor and the areas so rough and inaccessible that they are not reckoned on in the reforestation plan. This makes a total of 47.6 sq.mi. at present or very soon available for creating a municipal forest reserve.

For the purpose of more careful study and analysis the lower watershed was divided into four natural divisions, comprising the four main drainage basins. The first division is the Rock Creek, Webster Creek and Walsh Lake drainage basin. This area is all ready for the work of reforestation to be started and no future complications will arise because of logging operations. Also the absence of sawmills and logging railways greatly reduces the fire risk, and in case a fire should originate, the lay of the land is such that it could be checked easily and quickly. The soil is of good quality. The area is already well opened by former logging roads, skid roads and trails, so that all parts are easy of access, and planting operations can be easily conducted. The ground cover and weed growth are very light compared with their density on some of the other areas. There is not much down timber on this division and standing snags are few. This part of the lower watershed is near the town of Taylor, the works of the Denny Renton Clay & Coal Co. and the Columbia & Puget Sound R.R. This gives it a most favorable economic location, especially for the disposal of early products.

The second division comprises the drainage basin of William Creek and the Cedar River above Taylors Creek. A part of this has been logged off and the rest of the timber is now being cut, so on this part it is still possible to make an arrangement with the lumber company to leave seed trees for reforestation purposes. The soil is a poorer quality than that of the first division and dead stubs and down timber on the cut-over part are thicker. The fire risk is greater and the cost of preparing the area and the cost of reforestation will be greater; hence, it is recommended that the work be done on the first division before attempting to reforest the second division.

The third division comprises the entire drainage basin of Taylor Creek. The soil is of very fair quality and the future yield should be large. The soil here is covered with a thick, dense weed growth and is damp and stagnant. Snags and down timber are so thick as to make the cost of the work

FIG. 3. CUT-OVER AND BURNED-OVER LANDS, LOWER WATERSHED

very high in some places. The treatment recommended for the next few years is to allow the tract to be swept over by fires and as much of the dead material as possible consumed. When planting is decided upon, the tract should be burned clear the fall before and the planting done as quickly as possible the following spring and fall. Logging operations are still being conducted over a part of this area and it is not considered advisable to start reforestation at present.

The fourth division is a narrow strip comprising the lands draining into Cedar River below Taylor Creek, which is still well timbered although some is being logged off at the present time. The timber is now owned by different parties and

it is unknown when it will be logged. Nothing in the way of reforestation can be done here at present.

The lands in the first division lie at elevation of from 700 to 1400 ft., though 90% are below the 1200-ft. level. All this land except a small part back of the town of Taylor is open and rolling. It receives the sun all day, and such definite exposure as it may be said to have is south, which is, of course, the most favorable for rapid growth.

In the second division the lower part, near Cedar River, is level, while the hillsides back from the river have a south or southeastern exposure. That portion which lies south of Cedar River has a northwestern exposure, which is favorable for an even though not quite so rapid growth.

In the third division all the land which has now been logged off lies on a very gentle slope with a south or southwestern exposure. Of the hill sides that are not yet cut over

FIG. 4. STANDING TIMBER ON THE CEDAR LAKE WATERSHED

about three-quarters have a south or western exposure, while the rest are exposed to the north or northwest. Most of this land is about equally divided between a northern and southern exposure, though the slope is so gentle that sunlight is received during the greater part of the day in either case.

On the whole as regards exposure, over 75% of the entire area which can be reforested is so situated as to receive the best results. Fully two-thirds of the entire area lies below the 1600-ft. contour.

In the main the work of reforestation will be done with native species now growing under climatic conditions which it is self evident are particularly favorable. Rainfall is the most important of the climatic influences. The Douglas fir of the Pacific coast variety requires at least 40 in. of rainfall a year to obtain a fair growth, and above this measure the greater the annual precipitation the greater is the development of the tree. On the city watershed the annual precipitation, according to recorded measurements, varies from an average of 52 in. at Landsbury at an elevation of 540 ft., to an average of 112 in. at Cedar Lake, at an elevation of 1600 ft. The relative humidity is high and the winter temperature seldom below 0° F., while the summer temperature may be as high as 110°. The growing season lasts from April to well into October. The climatic conditions are particularly favorable to produce fine growths of Douglas fir. Frost, high winds and heavy snowfalls, all enemies to forest culture, are not elements to cause serious obstacles to reforesting any of the now logged off lands.

The present stand of trees in the surrounding forests seems to be particularly free from any signs of insect attacks, and this danger is one that need not be seriously considered. Attacks of fungi need not be dreaded. A stand managed on such a short rotation as 80 years should be in all respects too healthy for any fungus to obtain a foothold. Fungus attacks, with the exception of conk (polyporous) and occasional patches of trees affected with "butt rot" (P. schweinitzii) are also rather rare in the surrounding forest.

Douglas fir is especially adapted to the proposed system of management in its reproductive qualities. It is a prolific seed bearer, bearing some seed every year, with particularly heavy seed years about every third year. The seed is light and easily disseminated by the wind. This well adapts it to a system of regeneration by the leaving of seed trees, the least expensive of methods. The young nursery stock when set out makes a very rapid growth, which enables this species to be planted where there is a weed growth and still be able to forge ahead. It is a rapid growing tree both as to height

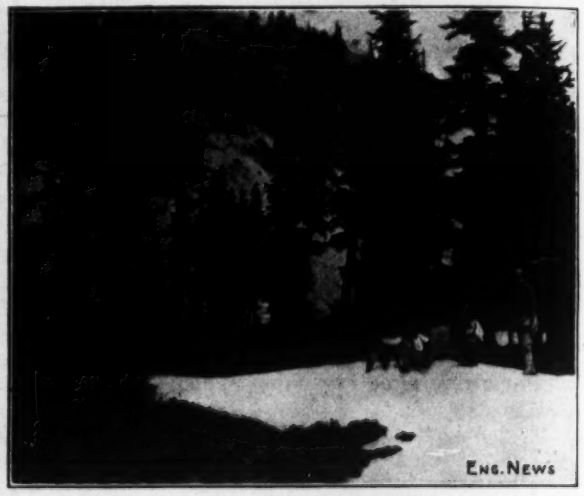

FIG. 5. VIEW IN THE UPPER WATERSHED, SURROUNDING CEDAR LAKE

and diameter, and this together with its ability to form dense, pure stands is what makes it so desirable a species.

Moreover, the products of Douglas fir formed at an early period can be utilized at a profit, which permits early revenue from thinning. Such thinnings will yield mine props, piles and some sawn timber. Owing to its adaptability to so many uses and to the fact that in the form of lumber it can be used for every ordinary purpose and its many excellent qualities, there is no doubt as to the future market for the product of this tree. All conditions point to it as being the one species that will yield the largest return and also be adaptable to the silvicultural point of management.

Western white pine (Pinus monticola) may prove of value to use in mixture with Douglas fir. Sitka spruce (Picea sitchensis) will probably be planted to a considerable extent on some of the areas. This species is particularly adapted to grow on certain of the more moist and swampy tracts, such as in the vicinity of Walsh Lake. After the first few years it makes a rapid height growth. It is tolerant while young and is able to get a good start under opposition and live until it can force its way up. Later it demands overhead light and does well in pure stands. The tree is a prolific seeder and once its growth is started on these areas its future reproduction will be an easy matter. It is also very free from fungus and insect attacks. The wood is valuable for cooperage, finishing, doors, packing cases and wood pulp. This species is a fast grower and will give a fairly large yield at the proposed length of rotation, and it will thrive where other species will not.

TABLE I. PER ACRE COST OF GROWING TIMBER UPON CEDAR RIVER WATERSHED UNDER MUNICIPAL OWNERSHIP

Average cost of stocking, $9.25 per acre. Administration and protection, 5c. per acre per annum. No taxes. No initial land cost. Interest rate, 4½%, compounded annually. Final yield taken as average of Qualities 1, 2 and 3, in proportion as they occur on the lower watershed, viz: 8% No. 1, 70% No. 2, and 22% No. 3 site quality.

(1) Age in years	(2) Cost of stocking, with accumulation of interest	(3) Administration and protection with interest	(4) Total cost of work at each decade	(5) Yield in ft. b.m. per acre	(6) Cost per M. ft. for growing
40	$53.80	$ 5.35	$59.15	9,600	$ 6.16
50	83.54	8.92	92.46	19,500	4.74
60	129.75	14.47	144.22	29,500	4.89
70	201.50	23.09	224.59	39,100	5.74
80	312.93	36.47	349.40	48,000	7.28
90	485.97	57.26	543.23	56,600	9.60
100	754.69	89.54	844.23	64,900	13.01

Black cottonwood (Populus trichocarpa) is probably the only native hardwood which may prove valuable for reforesting. It is also proposed to try out some of the eastern species, such as black walnut, hickory, yellow poplar, ash and Eastern white pine.

In estimating the yield of the reforested watershed the Douglas fir yield tables recently compiled by the U. S. Forest Service have been used. These tables are the result of a long extended series of observations and studies made in Western Washington and Oregon, and they are, therefore, reliable and based upon yields in the immediate vicinity of the proposed municipal forest. As one of the principal schemes of utilization is to consist of the marketing of the products obtained from thinnings, or intermediate cuts made during the rotation, much study was given this phase of the subject.

As to the future stumpage value of the lands of the watershed if properly forested and cared for there can be no question. The state of Washington at present leads all states in the amount of timber cut, the annual cutting averaging 4,500,000,000 ft., b.m., or a little over 10% of the entire cut of the whole United States. At such a rate it will not be long before all the forests in the state will be gone. The opening of the Panama Canal will probably increase the demand for west coast timber and increase the rapidity of cutting. When this supply is gone there will be no other virgin supply to fall back upon and there will be an inevitable rise in prices.

Douglas fir in the West presents a case almost analogous to that of the white pine of the Lake States, where stumpage prices increased in the 20 years, 1890 to 1910, from $5 per M. to $14.50 per M.

It is also absolutely certain that the timber lands now remaining in private hands will be cut over within the next few years, as the interest on initial cost and the taxes will not make it profitable to hold them for any very long period. The future supply of the west coast must then be obtained by reforestation. The cost of growing lumber for the market to private parties will obviously be greater than the cost to the city or other municipal corporations, which have no taxes to pay.

The estimated cost of stocking the lands of the Cedar River watershed previously referred to is $9.25 per acre, as shown in Table I. The cost of administration and protection is estimated at 5c. per acre per annum. There are no taxes to be paid; the initial land cost has been shown to be nothing, since it is already represented in standing timber owned by the city. The capital cost of stocking, administration and protection is figured on a 4½% interest basis, compounded annually. The final yields are based upon the proportions of the suitable soil areas as they occur in the lower watershed and upon the U. S. Forest Service yield tables previously referred to.

TABLE II. ITEMIZED STATEMENT OF ALL COSTS AND RECEIPTS PER AVERAGE ACRE AT EACH DECADE FOR ENTIRE LOWER WATERSHED

(1) Age of stand in years	(2) Summation of total cost of growing timber at each decade	(3) Summation of value of cash receipts from thinnings with interest	(4) Summation of actual cost of investment at each decade	(5) Stumpage value of final yield at each decade considered as an asset	(6) Profit or loss if final cut should be made at any particular period of 10 years	(7) Equivalent net annual revenue if final cut should be made at any particular 10-yr. period
40	$59.15		$59.15	$38.88	—$20.27	
50	92.46	$18.63	73.83	97.70	+23.87	$0.134
60	144.22	53.63	90.59	177.30	86.71	0.300
70	224.59	107.78	116.81	313.44	196.63	0.426
80	349.40	185.08	164.32	432.72	268.40	0.368
90	543.23	306.80	236.43	566.40	329.97	0.288
100	844.23	480.48	363.75	714.78	351.03	0.196
110	1,311.68	751.40	560.28	875.16	314.88	0.119
120	2,037.62	1,166.92	870.70	995.62	124.92	0.029

Note: For statement of items composing column 2, see columns 2 and 3, and compare with column 4 of Table I.

Table II shows a summarized statement of all the estimated costs and receipts per average acre at the end of each decade for the entire lower watershed An inspection of the table will show that the greatest net annual revenue per acre is at the expiration of 70 years. At 70 years the value of the standing timber is $313.44 per acre At that time the total value of the standing timber on the lower watershed would be $9,779,328.

The net profits to the city, after deducting the accumulated cost of stocking the land, of the management and protection of the timber, is $196.53 per acre, or a total of $6,135,000 for the area of 31,200 acres included in the lower watershed. This estimated net profit is obtained by the use of the most conservative stumpage values.

CONCLUSIONS

(1) The proposition of reforesting the Cedar River watershed is essentially practicable and there will be no difficulty in creating a new forest. Under proper management and with thorough protection against fire, the future yields should be at least equal to those forecasted in the U. S. Forest Service tables.

(2) The reforestation will prove of very great benefit to the water-supply of the city, and on that account alone the work may be justified.

(3) Values will be created where otherwise they would never exist. What is now a liability will be converted into an asset of very great value.

(4) The initial investment required is very small, and the total cost will be distributed over a number of years.

(5) The cost of the work will be returned to the city together with a profit estimated at from $6,000,000 to $9,000,000 by the expiration of 70 years, with a possibility of greatly increased profits at a longer period, particularly if the stumpage prices continue to increase in future as they have in the past.

NOTES FROM THE ENGINEERING SCHOOLS

UNIVERSITY OF MONTANA—At the last session of the state legislature, an act was passed providing that after July 1, 1913, the State University at Missoula, the College of Agriculture & Mechanic Arts at Bozeman, the School of Mines at Butte, and the Normal School at Dillon, should constitute the "University of Montana" with control and supervision invested in the State Board of Education. It is reported that all of the engineering courses maintained by the state, except the course in mining engineering at the School of Mines in Butte, will be concentrated in the college at Bozeman.

Prof. A. W. Richter, dean of the engineering department in the old state university, has been transferred to Bozeman and becomes dean of engineering there.

A State Hydro-Electric Power System will soon be built in Prussia, according to the London "Times" of Aug. 6, 1913. About $2,500,000 has been appropriated for the development of power at Hemfurt on the River Eder and at Helminghausen on the Dieme from existing reservoirs used with the Rhine-Hanover Canal. Later a third plant at Munden on the Fulda and Werra will be connected to the system. At Hemfurt the reservoir has a capacity of 1975 million gallons and a head of 135 ft.; there would be six 2300-kw. turbine generating units. At Helminghausen the reservoir has a capacity of 194 million gallons and a head of 114 ft.; here would be one 2000-kw. and two 600-kw. turbine units. At Munden there would be three 1200-kw. and three 450-kw. units. The Hemfurt and Helminghausen plants, from the start, would be tied in with the steam station of the Prussian Railways at Cassel and possibly with the municipal steam system in Cassel and Göttingen.

The state system, with 215 miles of 40,000-volt transmission line, would serve 13 counties in Prussia and three in Waldeck—about 2500 sq.mi., having a population of 600,000. Local companies and municipalities would intervene between small customers and the state system.

A Grease Cup Exploded on the driving rod of a Union Pacific R.R. locomotive at Evanston, Wyo., Aug. 19. The engineer and the fireman were injured.

A Fire on the Steel Tank Steamship "Burgomeister Hackman," lying at a pier at the Bush Terminal, Brooklyn, N. Y., Aug. 28, destroyed the vessel and its cargo of some 700,000 gal. of refined oil, valued at approximately $200,000. The ship had been chartered by the Vacuum Oil Co., 29 Broadway, New York City, to convey 750,000 gal. of oil to New Zealand and Australia. The oil, contained in 140,000 five-gallon cans, was being loaded into the hold by 100 men, when one of the men accidentally dropped a can of the oil down into the hold. This resulted in a severe explosion which was followed at intervals by many others. The ship was built five years ago by Siemers & Co., of Hamburg, Germany.

A Fire on the S.S. "Imperator" of the Hamburg-American Line, while lying at her dock in Hoboken, N. J., Aug. 28, did considerable damage. An official statement given out by the company says:

A fire broke out in the provision rooms of the steamship "Imperator" at about four a.m. The fire was confined absolutely to the compartment where the provision rooms are situated, and was under control after a few hours. No damage was done to the passenger accommodations, except in some of the companion ways of the second and third class, through which the hose lines were laid, but this damage was of no consequence, and can be repaired in a very short time.

It was necessary to flood that part of the ship in which the fire was situated, and she took a heavy list to starboard (about 12°). Sixteen lines of hose played water into the hold, from 4:30 to 8:30 a.m. The fire originated (from causes not yet determined) in a provision room on the after part of deck H, the deck just above the water line. The 1800 or so steerage passengers, who had not yet been transferred, were taken ashore and housed on the pier. In an effort to discover the actual conditions prevailing in the compartment containing the blaze, Second Officer Gobrecht lost his life. The "Imperator" entered the harbor at 7:30 p.m., Aug. 27, carrying 3193 passengers of all classes, the largest number ever transported across the Atlantic upon a single ship. The "Imperator" sailed on Aug. 30, as per schedule.

The San Francisco Municipal Ry. (Geary St. line) is reported by Wm. Dolge, of the City Supervisors' finance committee, to have earned a net profit of $19,364 in the seven months from Dec. 28, 1912, to July 31, 1913, after deduction for "actual expenses, depreciation, sinking fund and interest on bonds." On the basis of income, profit and loss statement for July, 1913, the estimated net profit for the coming year is $101,051. If it were a privately owned utility, he reports, state, federal and franchise taxes, license fees, attorney's services, insurance, etc., would reduce this profit to $52,578.

Municipal Street-Railway Lines to furnish transportation for the Panama-Pacific Canal Exposition were voted on Aug. 26, by the people of San Francisco. A $3,500,000 bond issue for this purpose was approved. The city has already embarked in municipal ownership with the Geary St. line, which was placed in operation Dec. 28, 1912. Bion J. Arnold, as consulting engineer, reported to the Board of Supervisors what lines were advisable and noted that the city must be prepared to shoulder the financial responsibility as private capital could not be expected to participate under the present franchise conditions stipulated by the city charter.

The Purchase of the Plant of the Des Moines Water Co. has been made the subject of a tentative agreement between the president of the company and James R. Hanna, Mayor of Des Moines. A valuation of $2,302,000 was placed on this plant by appraisal proceedings on Apr. 1, 1912. The water company immediately appealed from the decision insisting that it could not take less than $2,700,000 for the property plus the cost of extensions. The tentative agreement just made provides, as we understand it, for accepting the appraised price plus additions since Apr. 1, 1912. The agreement has yet to be approved by the trustees of the water company, by the city council and by the people of the city of Des Moines, according to a letter from Mayor Hanna dated Aug. 12.

The Hydro-Aëroplane Trip Around the British Coast, attempted by Harry G. Hawker, in response to the offer by the London "Daily Mail" of a prize of $25,000 for the successful completion of such a journey, ended Aug. 27 in an accident after nearly 70% of the required distance had been covered. After having traveled 1043 of the 1540 miles he fell into the sea 14 miles north of Dublin, Ireland, and was rescued, uninjured, by coast guards. The mishap was caused by one of his rubber-soled shoes slipping off the rudder bar as he was preparing to alight at Lough Shinny. The hydro-aëroplane got out of control and plunged downward 50 ft., falling into the sea a short distance from the shore, and was wrecked. In the first six stages of his flight, Hawker covered 836 miles in 853 min. flying time.

A Rear Collision occurred between the second section of the Bar Harbor express and the first section of the White Mountain express, on the New York, New Haven & Hartford R.R., near North Haven and about seven miles from New Haven, Conn., at 6.55 a.m., Sept. 2. Twenty-one persons were killed and many more injured. According to press dispatches the heavy traffic, due to the season, disarranged the schedule. The second section of the Bar Harbor express was held up on the main line, eight miles outside of New Haven, because of the pressure of traffic in the yards. The White Mountain express, running at about 35 mi. per hr., rammed the second section of the Bar Harbor express, demolishing the two rear Pullmans and overturning the third, causing it to roll down an embankment. No one seems to have been killed on the colliding train. The collision occurred on a clear stretch of track, with signals set against the following train, but the weather was foggy and the engineer claims that he saw no signals. The first official statement of the railroad was given out from the publicity office in the Grand Central Station at 8:40 a.m. and was as follows:

General Superintendent Woodward reports: White Mountain express, first section 95 due in New York at 7:32 a.m., ran into the rear end of second section 91, Bar Harbor express, due in New York 7:36, one mile north of North Haven, Conn., at 6:55 a.m. Two rear sleepers were damaged. Thirteen dead have been taken out. The first section consisted of seven cars drawn by engine 1337, A. B. Miller, engineer; conductor, Fowler. The second section had stopped for signal. The weather was extremely foggy.

The Chicago Garbage-Disposal Plan, by which the city agrees to purchase the plant of the Chicago Reduction Co., at a price to be determined by a board of appraisal, was settled finally at a special meeting of the City Council on Aug. 29. The agreement had been made some time previously, but since then the company had demanded that the contract should include provisions requiring the appraisers to take into consideration the "reasonable development expenses" incurred by the company in bringing the plant to its present condition and to consider also the peculiar adaptability of the site. As these specific requirements were not included in the original agreement, an ordinance had to be passed effecting the change, and as the company's contract expired Aug. 31, it was necessary to hold a special meeting. The ordinance was denounced vigorously, as was the dilatory method of the city officials in continually delaying consideration of the matter until action was rendered absolutely necessary by the expiration of the present contract with the company. The mayor suggested that the new clauses would have little effect upon the appraisal, but in reply to this it was asked why it should be necessary to rush these clauses through at a special meeting if they really had no effect. The company has insisted that if the matter was not settled by Aug. 31, it would cease to handle the garbage, and various alternative plans have been declared by the city authorities to be impracticable. The appraisers comprise Henry A. Allen (representing the city), Harold Almert (representing the company), and Leonard Metcalf, of Boston, as the third member.

Commission Government for Pennsylvania Cities of the Third Class was made compulsory by the legislature of 1913. The act affected 22 cities at the time of its passage and will include any accession to the list of third-class cities. Already a number of boroughs have initiated proceedings for a vote on the adoption of the act, which is open to any borough, town or township, or two or more such, having a population of 10,000 or more, all located in one county. All legislative powers of third-class cities are to be vested in a council composed of the mayor and four other members. The mayor is to serve for four years and the other councilmen for two years, and all are to be elected this fall and take office on Dec. 1, 1913, except in the case of mayors elected in 1911, who will serve until 1915. Appointed officers will also serve out their present terms. The first mayors and councilmen will have salaries of $500 a year for mayors and $250 for councilmen in cities of 15,000 or less, rising by steps to $3000 and $3500, respectively, for cities of 70,000 population and over. Any city council may fix the salaries of the mayor and members of the succeeding council at any figure between the limits named. The mayor is to be the head of the Department of Public Affairs, and the council is to choose from among its other members one to head each of these four additional departments: Accounts and Finance; Public Safety; Streets and Public Improvements; Parks and Public Property. Aside from a controller elected by popular vote for a period of four years, the council chooses all heads of executive departments, and in general has the power of appointing and dismissing all subordinate officers and employees, except as noted below in regard to employees under the city engineer. A board of health may be created by the council, or it may itself serve in that capacity, with the city clerk acting, ex-officio, as secretary of the board

in either case. The act makes no provision for civil service and demands no qualifications, in the way of education, training or fitness, for any officer, except that the city treasurer shall "be a competent accountant," the city solicitor a "person learned in the law and qualified to practice in the Supreme Court" of Pennsylvania, and the city engineer "a competent civil engineer." The city engineer seems to be given more power and dignity than the other executive officers under the council. He is to have "direction and control" of "the engineering matters of the city," and "shall appoint such number of assistants and employees as the council shall authorize by ordinance." No city department "shall employ or retain" any engineer save the city engineer except "with the previous assent of the council" and as regards water-works in any city where the water-works "is in the name of the commissioners of water-works." The act provides for the initiative and the referendum, but not for the recall. The 22 cities affected by the act are: Allentown, Altoona, Bradford, Carbondale, Chester, Corry, Easton, Erie, Harrisburg, Hazleton, Johnstown, Lebanon, McKeesport, Meadville, Oil City, New Castle, Pittston, Reading, Titusville, Wilkes-Barre, Williamstown, York. Since the act went into effect (up to Aug. 22), the borough of South Bethlehem has voted for commission government and the boroughs of Mount Carmel, Pottstown, Shamokin and Sunbury have voted against it. Under comparatively recent legislation, Pittsburgh and Scranton, cities of the second class, already have small single-chambered councils. Lancaster and four other minor cities are under special charters.

PERSONALS

Mr. T. Keaveny has been made Roadmaster of the Oregon-Washington R.R. & Navigation Co., at Colfax, Wash., succeeding Mr. H. J. Shaw, transferred.

Mrs. Cora B. Williams, widow of the late J. P. Willams, has been elected President of the Georgia, Florida & Alabama R.R., to succeed her late husband.

Mr. Jesse W. Lilienthal, a lawyer of San Francisco, Calif., has been elected President of the United Railroads, the San Francisco street railway system, to succeed Mr. Patrick Calhoun.

Mr. E. J. Parrish, Superintendent of Telegraph of the New York, Chicago & St. Louis R.R., has been promoted to be Division Superintendent at Fort Wayne, Ind., succeeding Mr. S. K. Blair, resigned.

Mr. R. R. Potter, formerly Assistant Editor of "Engineering News" has been promoted to be Assistant General Manager and Production Engineer of the Blood Bros. Machine Co., Kalamazoo, Mich.

Mr. J. L. Taylor, formerly Assistant Engineer of the Grand Rapids & Indiana Ry., Fort Wayne, Ind., has been appointed Assistant Division Engineer of the Pittsburgh, Cincinnati, Chicago & St. Louis Ry., Louisville, Ky.

Mr. G. M. Crownover, recently Master Mechanic of the Chicago Great Western Ry., at Oelwein, Iowa, has been promoted to be Superintendent of Motive Power, with office at Oelwein, succeeding Mr. J. G. Neuffer, resigned.

Mr. Frank Williams, for several years Superintendent of the South Works of the American Steel & Wire Co., Worcester, Mass., has been appointed Superintendent of all the works of the company at New Haven, Conn. He is succeeded at Worcester by Mr. Edward Hentz.

Mr. C. H. Sanderson, M. Am. Inst. E. E., recently Electrical Engineer, in charge of switchboard and power station design, Westinghouse Electric & Manufacturing Co., East Pittsburgh, Penn., has been appointed Chief Engineer of the Havana Electric Railway, Light & Power Co., Havana, Cuba.

Mr. W. S. Martin, recently Assistant General Manager of the Denver & Rio Grande R.R., Denver, Colo., has been promoted to be General Manager, succeeding Mr. E. L. Brown, who was recently made Vice-President in charge of operation of both the Denver & Rio Grande and the Western Pacific Ry., at San Francisco, Calif.

Mr. J. C. O'Neill, formerly an Engineer on the construction of the Chicago, Milwaukee & Puget Sound Ry., has been appointed Chief Engineer of the Butte, Wisdom & Pacific Ry., a new railway which is being extended from Butte into southwestern Montana. The railway is to be 128 miles long and it is reported construction work is soon to be begun.

Mr. C. H. Stevens has been appointed Chief Draftsman of the new Department of City Transit, Philadelphia, Penn., at a salary of $2700 per annum. Mr. S. G. Griffith has been appointed Assistant Engineer of the same department at a salary of $2200 per annum; and Mr. H. B. Hughes has been appointed Assistant Engineer at a salary of $2000 per annum.

Mr. Joseph Mayer, M. Am. Soc. C. E., Principal Assistant Engineer, Quebec Bridge Commission, Montreal, Que., was the designer and Chief Engineer of construction of the superstructure of the famous Poughkeepsie Bridge over the Hudson River and not the late Robert Escobar, as stated in our issue of Aug. 7, p. 276. Mr. Escobar's connection with the Union Bridge Co. is given more in detail in a letter by Mr. Charles Macdonald, Past-President, Am. Soc. C. E., elsewhere in this issue.

Mr. Howard M. Biscoe, Traffic Manager of the Boston & Albany R.R., at Boston, Mass., has been elected Vice-President in charge of operation to succeed Mr. James H. Hustis, who became President in charge of operation of the New York, New Haven & Hartford R.R., on Sept. 1. Mr. Biscoe was born in Westboro, Mass., in 1869. He graduated from Yale University in 1892 and entered the service of the Boston & Albany R.R. the same year as a clerk in the ticket auditor's office in Boston. He has been Traffic Manager since 1910.

Mr. Hugh L. Cooper, M. Am. Soc. C. E., Consulting Engineer, New York City, and Chief Engineer of the recently completed Keokuk plant of the Mississippi River Power Co., has been further honored by having the lake, covering some 700 sq. mi., which is formed by the dam across the Mississippi River, named Lake Cooper. To perpetuate the memory of our notable engineers by associating their names with the products of their energy and skill is an excellent practice and will do much to keep straight the record of historical engineering enterprises. Mr. F. H. Newell, M. Am. Soc. C. E., Director of the U. S. Reclamation Service, enjoys the distinction of having a town in South Dakota named for him, so at least the names of two of our present-day engineers will become a part of the geography of the United States.

OBITUARY

William Carpenter Smith, Chief Engineer of Maintenance-of-way of the Northern Pacific Ry., St. Paul, Minn., died Aug. 26, at his home in St. Paul. He had been ill for several months. Mr. Smith was born July 30, 1865, at St. Cloud, Minn. He graduated in civil engineering from the Minnesota State University in 1890, and soon after entered the service of the St. Paul & Northern Pacific Ry., as Resident Engineer. From 1893 to 1897 he was Assistant Engineer of the Northern Pacific Ry., at Duluth, Minn., and later held the same position for two years at Jamestown, N. D. From 1899 to 1902 he was Assistant Engineer in charge of construction work. In 1902 he was promoted to be Division Engineer at Livingston, Mont., which position he held for five years, most of which he was stationed at St. Paul. In 1907 he was promoted to be Engineer of Maintenance-of-way, and three years later to be Chief Engineer of Maintenance-of-way.

Francis Valentine T. Lee, M. Am. Inst. E. E., Consulting Engineer, of Victoria, B. C., and London, England, died in Victoria, Aug. 17. He was born in 1870 in Winchester, England. His father was a British Army officer. Mr. Lee came to the United States when 17 years old and was employed by an electrical supply company in New York City. Here he met the late Dr. F. A. C. Perrine, at that time Professor of electrical engineering at the Leland Stanford, Jr., University, who induced him to go to California. After completing a course at the university he was employed in engineering work on the construction of several hydro-electric plants, which are now a part of the property of the Pacific Gas & Electric Co. During this time he was associated with John Martin & Co. In 1906 he entered the employ of the Pacific Gas & Electric Co., where he remained until 1910. Since then he had practiced extensively on the Pacific coast as a consulting engineer. Mr. Lee was also a member of the Institution of Electrical Engineers, of Great Britain.

Franklin Allen Hinds, M. Am. Soc. C. E., whose death was noted in our issue of last week, was educated at Jefferson County Institute, New York, and after a short period spent in Portland, Ore., at the Sheffield Scientific School, Yale University. After leaving college he was employed for a year by an engineer in New York City, whose specialty was landscape gardening and drainage. In 1868 he was engaged in surveys for the Black River & St. Lawrence R.R., which was first laid with wooden rails. It was years afterward recon-

structed and is now the Carthage and Adirondack branch of the New York Central Lines. From this work he went to the Carthage, Watertown & Sacketts Harbor R.R., as noted in our issue of last week.

In 1874-'75, Mr. Hinds laid out the Thousand Islands Park and later nearly all the parks on the St. Lawrence River. In 1880, after serving as City Engineer of Watertown, he took into partnership H. C. Hodgkins, M. Am. Soc. C. E., and the firm of Hinds & Hodgkins were engaged in the design and construction of numerous water-works plants in the East. In 1883 the firm was increased by the addition of J. F. Moffett, and the firm of Hinds, Moffett & Co., continued in the construction of water-works. A few years after Mr. Hinds sold his share of the partnership to the other members of the firm, which became Moffett, Hodgkins & Clarke. In 1887-'88 Mr. Hinds was engaged in the construction of the Ontario Paper Mills, near Brownsville. In 1889 he formed a partnership with E. A. Bond, M. Am. Soc. C. E., later State Engineer of New York.

Mr. Hinds had one of those lovable dispositions that made friends wherever he went, and he never lost one. The impress of his life's work is upon all the public institutions of the city he loved and served faithfully for so many years. [Contributed by Mr. H. C. Hodgkins, Consulting Engineer, Syracuse, N. Y.]

ENGINEERING SOCIETIES

COMING MEETINGS

AMERICAN BOILER MANUFACTURERS' ASSOCIATION.
Sept. 1-4. Annual convention at Cleveland, Ohio. Secy., J. D. Farasey, 37th St., Cleveland, Ohio.

INTERNATIONAL ASSOCIATION OF FIRE ENGINEERS.
Sept. 1-6. Annual convention at New York City. Secy., Jas. McFall, Roanoke, Va.

ROADMASTERS' AND MAINTENANCE OF WAY ASSOCIATION.
Sept. 8-12. Annual convention at Chicago, Ill. Secy., L. C. Ryan, Chicago and North Western Ry., Stirling, Ill.

AMERICAN CHEMICAL SOCIETY.
Sept. 8-12. Annual meeting at Rochester, N. Y. Secy., Chas. L. Parsons, Durham, N. H.

AMERICAN ELECTROCHEMICAL SOCIETY.
Sept. 9-11. The 24th general meeting at Denver. Secy., Prof. Jos. W. Richards, Lehigh University, South Bethlehem, Penn.

NEW ENGLAND WATER-WORKS ASSOCIATION.
Sept. 10-12. Annual convention at Philadelphia, Penn. Secy., Willard Kent, Narragansett Pier, R. I.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Sept. 9-13. Annual meeting at Colorado Springs, Colo. Secy., S. M. Gunn, 755 Boylston St., Boston.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 9-12. Annual convention at Ottawa, Ont. Secy., A. P. Dane, Reading, Mass.

NATIONAL PAVING BRICK MANUFACTURERS ASSOCIATION.
Sept. 17-18. Tenth annual meeting at Cleveland, Ohio. Secy., W. P. Blair, 824 Engineers' Building, Cleveland, Ohio.

INTERNATIONAL CONGRESS OF REFRIGERATION.
Sept. 17-Oct. 1. Third congress, at Chicago, Ill.
Secy., J. F. Nickerson, Chicago.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 22-26. Annual convention at Pittsburgh, Penn. Secy., I. D. Israel, 29 W. 39th St., New York City.

AMERICAN MINE SAFETY ASSOCIATION.
Sept. 22-24. Annual meeting at Pittsburgh, Penn. Chairman, H. M. Wilson, 40th and Butler Sts., Pittsburgh, Penn.

AMERICAN ROAD CONGRESS.
Sept. 29-Oct. 5. Annual convention at Detroit, Mich.
Gen. Secy., J. C. Pennybacker.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 7-10. Annual convention at Wilmington, Del. Secy., Geo. H. McGovern, Chamber of Commerce, Wilmington, Del.

RAILWAY SIGNAL ASSOCIATION.
Oct. 14-16. Annual meeting at Nashville, Tenn. Secy., C. C. Rosenberg, Times Building, Bethlehem, Penn.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 13-17. Annual convention at Atlantic City, N. J. Secy., H. C. Donecker, 29 West 39th St., New York City.

AMERICAN GAS INSTITUTE.
Oct. 15-17. Annual meeting at Richmond, Va. Secy., Geo. G. Ramsdell, 29 West 39th St., New York City.

AMERICAN MINING CONGRESS.
Oct. 20-25. Annual convention at Philadelphia, Penn. Secy., J. Calibreath, Majestic Building, Denver, Colo.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 21-23. Annual convention at Montreal, Que. Secy., C. A. Lichty, 319 North Waller Ave., Chicago, Ill.

NATIONAL ASSOCIATION OF RAILWAY COMMISSIONERS.
Oct. 28. Annual Convention at Washington, D. C. Secy., Wm. H. Connolly, Interstate Commerce Commission, Washington, D. C.

American Chemical Society—The 48th meeting will be held at Rochester, N. Y., Sept. 8-12. The program is divided as follows: (1) Agricultural and Food Chemistry; (2) Organic Chemistry; (3) Biological Chemistry; (4) Pharmaceutical Chemistry; (4) India Rubber Chemistry; (5) Industrial Chemists and Chemical Engineers; (6) Physical and Inorganic Chemistry; (7) Fertilizer Chemistry; (8) Water, Sewage and Sanitation. An idea of the scope of this convention will be had from the fact that 168 papers are scheduled for presentation before the above divisions.

National Paving Brick Manufacturers Association—The tenth annual meeting, at Cleveland, Ohio, Sept. 17-18, will include a conference of city engineers and other officials and paving contractors. Committees representing the American Society of Municipal Improvements and the American Society for Standardizing Paving Specifications, will meet at Cleveland, Sept. 15-16, to visit brick-making plants and then attend the meeting.

The mornings of Sept. 17 and 18 will be devoted to the business sessions of the Association, and at the dinner on Sept. 18 there will be opportunity for discussions. As both the city of Cleveland and Cuyahoga county are increasing their mileage of brick pavements, visits of inspection will be made to construction work in progress as well as to completed work. It is stated that these visits will include city streets with brick paving laid 19 years ago and which have not required any expenditure for maintenance due to wear and tear.

Central States Water-Works Association—The seventeenth annual convention was held at Cedar Point on Lake Erie (near Sandusky, Ohio), on Aug. 26 to 28, with T. C. Beardsley, of Cleveland, Ohio, in the chair. Upward of 75 members and guests were present, four papers were read, some seven or eight manufacturers had exhibits, there was a visit to the Sandusky filtration plant and a dinner was given by the supply men.

J. C. Martin, of Wilmington, Del., prominent in the electric-light circles of Ohio, was advanced from the vice-presidency to the presidency, and F. W. Collins, of Manistee, Mich., was made vice-president. R. P. Bricker, of Shelby, Ohio, was reëlected as secretary and A. W. Inman, of Massillon, Ohio, was reëlected treasurer. Wheeling, W. Va., was chosen as the next meeting place, the exact date to be fixed later.

During the convention there was considerable discussion of the proposition to become a chapter or local section of the American Water-Works Association. A number favored the plan, but some feared that the loss of identity and of opportunity for the men connected with the smaller works would more than offset any gain from affiliation. Edward Bartow, of the Illinois State Water Survey, who is a member of a committee of the American Water Works Association, charged with formulating a plan for taking in other societies, forming chapters, etc., read copies of tentative reports on the subject, recently drawn for presentation to the larger association. It was finally voted to refer the matter of affiliation to the executive committee of the Central States Water Works Association, plus one member from each state not represented on that committee, for report a year hence.

A paper on water-works valuation, by Philip Burgess, of Columbus, Ohio, gave some facts regarding valuations of the private works of Chillicothe and Texarkana, Ohio, in which he represented the city. Most of the paper was taken up with an outline of the various methods of estimating cost and depreciation. Mr. Martin, in discussing the paper, argued for valuation on the basis of earning power and urged the water-works men (private plants) of Ohio to stand together in all proceedings before the Ohio Public Service Commission. Mr. Burgess stated that an Ohio law, which went into effect in August, 1903, provides that valuations for either purchase or rate making shall be on the reproduction-new basis and shall consider the question of franchise and good will, but fails to state what account shall be taken of these.

A short paper by T. F. Hynes described the use of "Water Motors for Vacuum Heating Systems." Mr. Martin pointed out that unless the water so employed could be used over again the practice was likely to be uneconomical and urged that the cost of water-motor power should be compared with that of electric or gas power before deciding on the use of the former. He said that where water is sold at flat rates there is a strong tendency for consumers to use water motors to their own personal saving, but at the expense of the water department or company.

A short paper by J. B. Merriam dealt in an elementary way with "The Gas Engine in Its Adaptation to the Industries." W. J. Springborn, Director of Public Service, Cleveland, Ohio, spoke warmly in favor of municipal ownership in general and painted a glowing picture of the success of the Cleveland water-works under municipal ownership.